# THE DESIGN OF HIGHWAY BRIDGES

# OF STEEL, TIMBER AND CONCRETE


BY

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# PREFACE TO SECOND EDITION

The increase in live loads due to the extensive use of heavy motor trucks, tractors and traction engines, and the increased use of reinforced concrete in building highway bridges have made it necessary to rewrite this book. The scope of the work has been extended so that the book now covers the design of concrete and timber highway bridges as well as steel highway bridges. The design of both the superstructure and the substructure of highway bridges is discussed in detail. The discussion covers all the details of constructing highway bridges, including the calculation of the stresses, the design, the estimate, the contract and the erection and construction.

The same size of type page and size of type as are used in the author's "Structural Engineers' Handbook" are used in this book.

The book is divided into four parts and in addition has three appendices.

Part I covers the calculation of the stresses in bridge trusses and in bridge portals and other details. Both algebraic and graphic methods of calculating stresses in bridge trusses are described in detail. Stresses are calculated in bridge trusses for both equal joint loads and for wheel concentrations. Chapter VII contains the solutions of 27 problems in the calculation of stresses in bridge trusses. Influence diagrams are developed for girders and trusses in this chapter. Part I covers the first course in the calculation of stresses in bridges given in the author's classes.

Part II covers the design of steel and timber highway bridges. The design of steel highway bridges is divided into beam bridges, low truss bridges, plate girder bridges and high truss bridges. The design of bridge floors is considered in detail, and data are given for the design of steel highway bridges. The chapter on timber bridges includes timber trestles as well as timber truss bridges. The design of a beam bridge, a plate girder bridge, a low truss bridge and a high truss bridge are worked out in detail.

Part III covers the design of reinforced concrete highway bridges and foundations. The formulas for calculating the stresses in reinforced concrete structures are developed. The different types of reinforced concrete beam, girder, arch and reinforced concrete trestle bridges are discussed, and working plans are given for all types. Designs of the different types of bridge are worked out in detail. Algebraic and graphic solutions are given for the elastic arch. An influence diagram solution is also given for the elastic arch. Abutments and piers for steel and concrete bridges are discussed and many examples of structures are given. The different types of culverts are described in detail with examples of plans. The overflow bridge, which has been developed to meet a special need in localities subject to excessive flood flow, is described and examples are given.

Part IV covers the details of bridge design, bridge contracting, estimates and costs, and bridge erection and construction. The discussion in Part IV together with Appendix I, "General Specifications for Steel Highway Bridges," and Appendix II, "General Specifications for Concrete Bridges and Foundations," covers in detail the design and erection of steel and concrete highway bridges. Structural tables of especial value in the design of steel highway bridges are given in Appendix III.

The very rapid advance in the design of highway bridges is mainly due to the excellent work done by the various state highway commissions, and more recently the work done by the U. S. Bureau of Public Roads. The author wishes to express his appreciation for the uniform courtesy of the various commissions and bureaus in furnishing plans and specifications. The author is under especial obligations to Mr. Clifford Older, bridge engineer of the Illinois Highway Commission; Mr. M. W. Torkelson, bridge engineer of the Wisconsin Highway Commission; Mr. J. H. Ames, and Mr. E. F. Kelly, bridge engineers of the Iowa Highway Commission; and Mr. C. V. Dewart, bridge engineer of the Michigan State Highway Department, for furnishing plans, specifications and data, without which this book could not have been written. The writer also wishes to thank the U. S. Bureau of Public Roads for furnishing the blue prints of their standard plans, which are reproduced in this book.

Credit is due Professor W. C. Huntington, University of Colorado, for assistance in preparing drawings and making calculations, to Professor R. S. Wallis, Missouri School of Mines, for assistance in preparing drawings; to H. C. Ford, for assistance in preparing drawings, and to C. L. Eckel, Assistant Professor of Civil Engineering in the University of Pennsylvania, for assistance in making calculations, preparing drawings and reading proof.

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# DESIGN OF HIGHWAY BRIDGES OF STEEL, TIMBER AND CONCRETE

**Introduction.**—Highway bridges are built (1) of steel; (2) of steel or iron and timber; (3) of timber; (4) of stone or concrete masonry, and (5) of reinforced concrete.

**Steel Bridges.**—Steel bridges may for convenience be divided into (a) beam bridges; (b) plate girder bridges; (c) low truss bridges, and (d) high truss bridges. Truss bridges are made with pin-connected joints, "pin-connected," or with riveted joints, "riveted."

**Combination Bridges.**—Combination bridges have timber upper chords, posts and struts, and steel or iron tension members. Combination bridges are commonly made with the Pratt type of truss, and may have parallel or inclined chords. Combination bridges are used only where timber is cheap, and steel and iron are relatively expensive.

**Timber Bridges.**—Timber is used for trestles and truss bridges, and occasionally for culverts. The Howe truss is usually made with timber upper and lower chords and diagonal struts, the vertical ties being steel or iron rods. Timber bridges are used for temporary structures and for locations where timber is available and transportation of materials for more permanent structures is difficult or very expensive.

**Masonry Bridges.**—Arch bridges were formerly made of stone masonry or plain concrete. Masonry bridges, when properly designed and constructed, are permanent structures.

**Reinforced Concrete Bridges.**—Reinforced concrete is now quite generally used for highway bridges, trestles, arches and culverts. Reinforced concrete structures can be built in locations where it would not be possible to build ordinary masonry arches, and can usually be built for less money. Reinforced concrete structures require great care in construction, and when so constructed are permanent.

The calculation of stresses in steel bridges is considered in Part I. While the discussion is concerned primarily with highway bridges, the calculation of stresses due to wheel concentrations is briefly considered.

The solutions of 27 problems in the calculation of stresses in bridge trusses are given.

The design of steel and timber highway bridges is considered in Part II. The discussion of types of structures, widths of roadway, types of floor covering and live loads given in Part II, applies to reinforced concrete bridges as well as to steel and timber bridges. Designs of steel bridges of the different types are worked out in detail, and many plans of actual structures are given. Specifications are given for the design and construction of timber bridges and trestles.

The design of reinforced concrete bridges and foundations is considered in Part III. The discussion includes a brief resume of the theory of reinforced concrete design. The stresses in the elastic arch are calculated by algebraic and graphic methods, and by influence diagrams. The design of abutments and piers and retaining walls are considered. Designs of concrete bridges of the different types are worked out in detail, and many plans of actual structures are given.

The principles of location and design, the estimate of cost, and the erection and construction of highway bridges are considered in Part IV.

Specifications for the design of steel highway bridges are given in Appendix I.

Specifications for the design of concrete highway bridges and foundations are given in Appendix II.

Tables for the design of highway bridges are given in Appendix III.

# PART I.

# STRESSES IN STEEL BRIDGES.

## CHAPTER I.

### METHODS FOR THE CALCULATION OF STRESSES IN FRAMED STRUCTURES.

**Introduction.**—Structures are acted upon by external forces, called loads, the weight of the structure, the reactions of the supports, the force of the wind, etc. These external forces are held in equilibrium by internal forces called stresses. If a straight member is acted upon at its ends by two equal external forces in the direction of its length, equilibrium at any right section of the member will be maintained by internal forces called stresses acting on opposite sides of the section, equal in amount, but opposite in direction to the external forces. When the external forces tend to elongate the member, the stress is tension; when the external forces tend to shorten the member, the stress is compression; while when the external forces tend to shear the member off, the stress is shear. Strain is the deformation caused by stress; the ratio of stress to strain being equal to a quantity, usually a constant, called the modulus of elasticity. Compressive stresses will be considered as positive stresses, while tensile stresses will be considered as negative stresses.

Forces acting in a plane are called coplanar forces. Coplanar forces alone will be considered in this chapter. Forces meeting in a common point are called concurrent forces, while forces which do not all meet in a common point are called non-concurrent forces.

**Representation of Forces.**—A force is determined when its magnitude, line of action and direction are known. It may be represented algebraically by stating the number of units in the force, by giving the coördinates of a point in the line of action of the force, and by stating the angle made by the line of action of the force with a line of reference; or it may be represented graphically in magnitude by the length of a line, in line of action by the position of the line, and in direction by an arrow placed on the line pointing in the direction in which the force acts.

**Equilibrium.**—Statics considers forces at rest, and therefore in equilibrium. To have static equilibrium in any system of forces there must be neither translation nor rotation, and the following conditions must be fulfilled for coplanar forces.

$$\Sigma \text{ horizontal components of forces} = 0 \qquad (1)$$

$$\Sigma \text{ vertical components of forces} = 0 \qquad (2)$$

$$\Sigma \text{ moments of forces about any point} = 0 \qquad (3)$$

Problems in statics can be solved graphically or algebraically. The determination of the reactions of a simple framed structure usually requires the use of equations (1), (2) and (3). Having completely determined the external forces the internal stresses may be obtained by the use of equations (1) and (2) (resolution), or equation (3) (moments). These equations may be solved by algebra or graphics.

There are, therefore, four methods of calculating stresses, viz.:

$$\text{Resolution of Forces}\begin{cases}\text{Algebraic Method.}\\ \text{Graphic method.}\end{cases}$$

$$\text{Moments of Forces}\begin{cases}\text{Algebraic Method.}\\ \text{Graphic Method.}\end{cases}$$

The stresses in any simple framed structure can be calculated by using any one of the four methods. However, there is usually one method best suited to the solution of each particular problem.

**RESOLUTION.**—In calculating the stresses in a truss by resolution the fundamental equations for equilibrium for translation

$$\Sigma \text{ horizontal components of forces} = 0 \qquad (1)$$

$$\Sigma \text{ vertical components of forces} = 0 \qquad (2)$$

are applied to the structure at the joints or to sections.

**Force Triangle.**—The resultant, $R$, of the two forces $P_1$ and $P_2$ meeting at the point a in Fig. 1 is represented in magnitude and direction by the diagonal, $R$, of the parallelogram $a$–$b$–$c$–$d$. The combining of the two forces $P_1$ and $P_2$ into the force $R$ is termed composition of forces. The reverse process is called resolution of forces.

The value of $R$ may be found algebraically from the equation

$$R^2 = P_1^2 + P_2^2 + 2P_1P_2 \cos\theta$$

It is not necessary to construct the entire force parallelogram as in (a) Fig. 1, the force triangle (b) below or (c) above the resultant $R$ being sufficient.

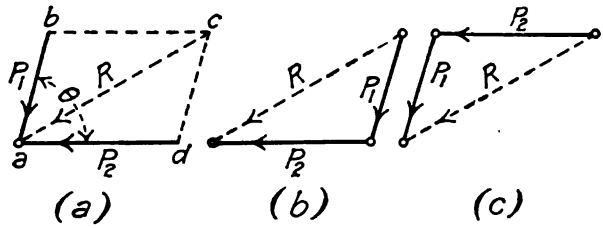



FIG. 1.

If only one force together with the line of action of the two others be given in a system containing three forces in equilibrium, the magnitude and direction of the two forces may be found by means of the force triangle.

If the resultant $R$ in Fig. 1 is replaced by a force $E$ equal in amount but opposite in direction, the system of forces will be in equilibrium, (a) or (b) Fig. 2. The force $E$ is the equilibrant of the system of forces $P_1$ and $P_2$.

It is immaterial in what order the forces are taken in constructing the force triangle, as in Fig. 2, as long as the forces all act in the same direction around the triangle. The force triangle is the foundation of the science of graphic statics.

**Force Polygon.**—If more than three concurrent forces (forces which meet in a point) are in equilibrium as in (a) Fig. 3, $R_1$ in (b) will be the resultant of $P_1$ and $P_2$, $R_2$ will be the resultant of $R_1$ and $P_3$, and will also be the equilibrant of $P_4$ and $P_5$. The force polygon in (b) is therefore only a combination of force triangles. The force polygon for any system of forces may be constructed as follows: Beginning at any point draw in succession lines representing in magnitude and direction the given forces, each line beginning where the preceding one ends. If the polygon closes, the system of forces is in equilibrium, if it does not close the line joining the first and last

points represents the resultant in magnitude and direction. As in the case of the force triangle, it is immaterial in what order the forces are applied as long as they all act in the same direction around the polygon. A force polygon is analogous to a traverse of a field in which the bearings

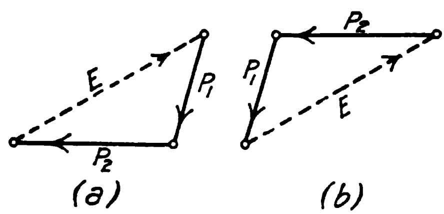



FIG. 2.

and the distances are measured progressively around the field in either direction. The conditions for closure in the two cases are also identical.

It will be seen that any side in the force polygon is the equilibrant of all the other sides, and that any side reversed in direction is the resultant of all the other sides.

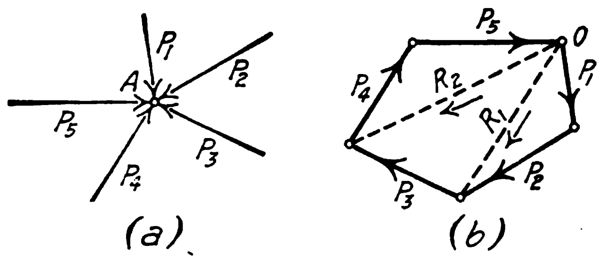



FIG. 3.

**Equilibrium of Concurrent Forces.**—The necessary condition for equilibrium of concurrent coplaner forces therefore is that the force polygon close. This is equivalent to the algebraic condition that $\Sigma$ horizontal components of forces = 0, and $\Sigma$ vertical components of forces = 0. If the system of concurrent forces is not in equilibrium, the resultant can be found in magnitude and direction by completing the force polygon. The resultant of a system of concurrent forces is always a single force acting through their point of intersection.

**Algebraic Resolution.**—In calculating the stresses in a truss by algebraic resolution, the fundamental equations for equilibrium, (1) and (2), for translation are applied (a) to each joint, or (b) to the members and forces on one side of a section cut through the truss.

(a) *Forces at a Joint.*—The reactions having been found, the stresses in the members of the truss shown in Fig. 4 are calculated as follows: Beginning at the left reaction, $R_1$, we have by applying equations (1) and (2)

$$1\text{-}x \cdot \sin\theta - 1\text{-}y \cdot \sin\alpha = 0 \tag{4}$$

$$1\text{-}x \cdot \cos\theta - 1\text{-}y \cdot \cos\alpha - R_1 = 0 \tag{5}$$

The stresses in members 1–$x$ and 1–$y$ may be obtained by solving equations (4) and (5). The direction of the forces which represent the stresses in amount will be determined by the signs of the results; if compressive stresses are assumed as positive, tensile stresses will be negative. Arrows pointing toward the joint indicate that the member is in compression; arrows pointing away from the joint indicate that the member is in tension. The stresses in the members of the truss at the remaining joints in the truss are calculated in the same way.

The direction of the forces and the kind of stress can always be determined by sketching in the force polygon, for the forces meeting at the joint as in (c) Fig. 4.

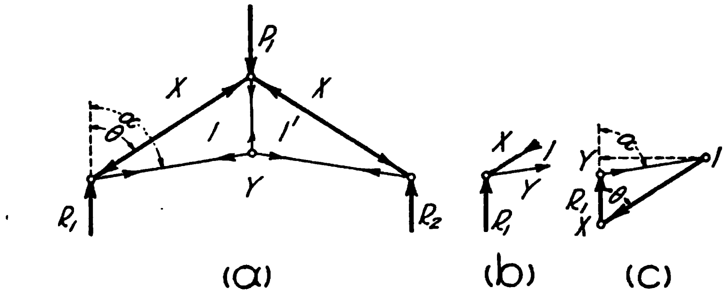


FIG. 4.

It will be seen from the foregoing that the method of algebraic resolution consists in applying the principle of the force polygon to the external forces and internal stresses at each joint.

Since we have only two fundamental equations for translation (resolution) we can not solve a joint if there are more than two forces or stresses unknown.

Where the lower chord of the truss is horizontal as in Fig. 5, we have by applying fundamental equations (1) and (2) to the joint at the left reaction

$$1\text{-}x = +R_1 \cdot \sec\theta \tag{6}$$

$$1\text{-}y = -R_1 \cdot \tan\theta \tag{7}$$

the plus sign indicating compression and the minus sign tension. Equations (6) and (7) may be obtained directly from force triangle (c).

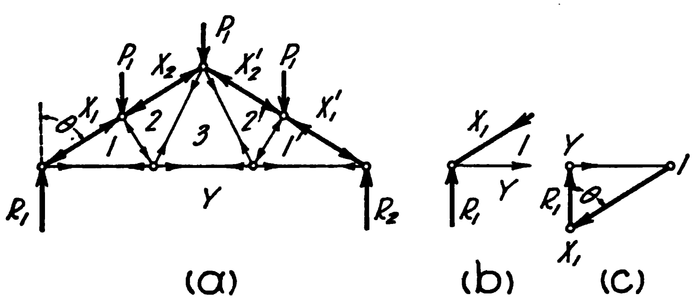


FIG. 5.

(b) *Forces on One Side of a Section.*—The principle of resolution of forces may be applied to the structure as a whole or to a portion of the structure.

If the truss shown in Fig. 6 is cut by the plane $A$–$A$, the internal stresses and external forces acting on either segment, as in (b) will be in equilibrium. The external forces acting on the cut members as shown in (b) are equal to the internal stresses in the cut members and are opposite in direction.

Applying equations (1) and (2) to the cut section

$$3\text{-}y + 2\text{-}3 \cdot \cos\alpha - 2\text{-}x \cdot \sin\theta = 0 \tag{8}$$

$$2\text{-}3 \cdot \sin\alpha - 2\text{-}x \cdot \cos\theta + R_1 - P_1 = 0 \tag{9}$$

Now, if all but two of the external forces are known, the unknowns may be found by solving equations (8) and (9). If more than two external forces are unknown the problem is indeterminate as far as equations (8) and (9) are concerned.

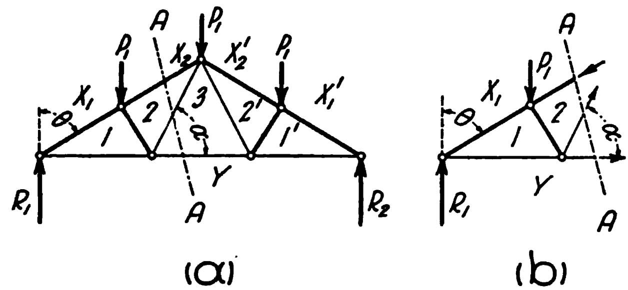



FIG. 6.

In the Warren truss in Fig. 7 the stresses at a joint may be calculated by completing the force polygon as at the left reaction in (b) Fig. 5. Applying equations (1) and (2) to a section as in (c)

$$2\text{–}x + 2\text{–}3\cdot\sin\theta - 3\text{–}y = 0 \tag{10}$$

$$-\,2\text{–}3\cdot\cos\theta - P + R_1 = 0 \tag{11}$$

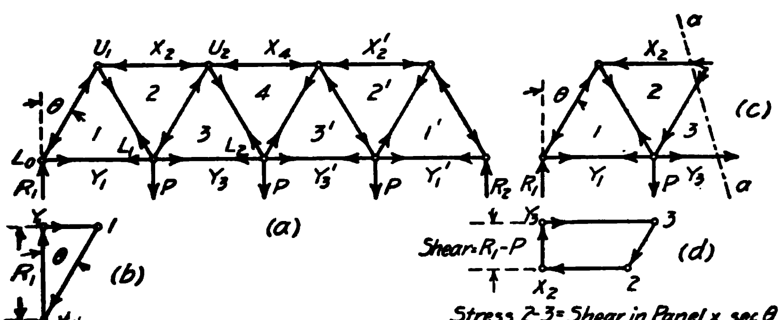



FIG. 7.

Now, $R_1 - P$ = shear in the panel. Therefore the stress in 2–3 = $-(R_1 - P)\sec\theta$ = shear in panel $\times$ sec $\theta$. This analysis leads directly to the method of coefficients as explained in detail in Chapter III.

**Graphic Resolution.**—In Fig. 8 the reactions $R_1$ and $R_2$ are found by means of the force and equilibrium polygons as shown in (b) and (a). The principle of the force polygon is then applied to each joint of the structure in turn. Beginning at the joint $L_0$ the forces are shown in (c), and the force triangle in (d). The reaction $R_1$ is known and acts upward, the upper chord stress 1–$x$ acts downward to the left, and lower chord stress 1–$y$ acts to the right closing the polygon. Stress 1–$x$ is compression and stress 1–$y$ is tension, as can be seen by applying the arrows to the members in (c). The force polygon at joint $U_1$ is then constructed as in (f). Stress 1–$x$ acting toward joint $U_1$ and load $P_1$ acting downward are known, and stresses 1–2 and 2–$x$ are found by completing the polygon. Stresses 2–$x$ and 1–2 are compression. The force polygons at joints $L_1$ and $U_2$ are constructed, in the order given, in the same manner. The known forces at any

joint are indicated in direction in the force polygon by double arrows, and the unknown forces are indicated in direction by single arrows.

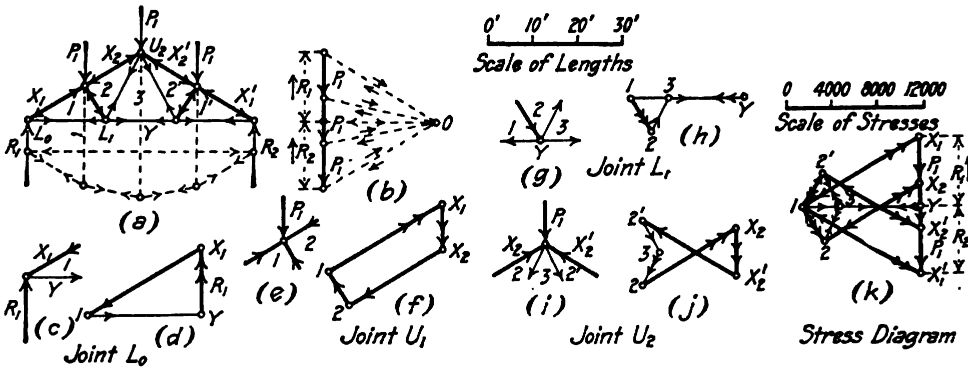



FIG. 8.

The stresses in the members of the right segment of the truss are the same as in the left, and the force polygons are, therefore, not constructed for the right segment. The force polygons for all the joints of the truss are grouped into the stress diagram shown in (k). Compression in

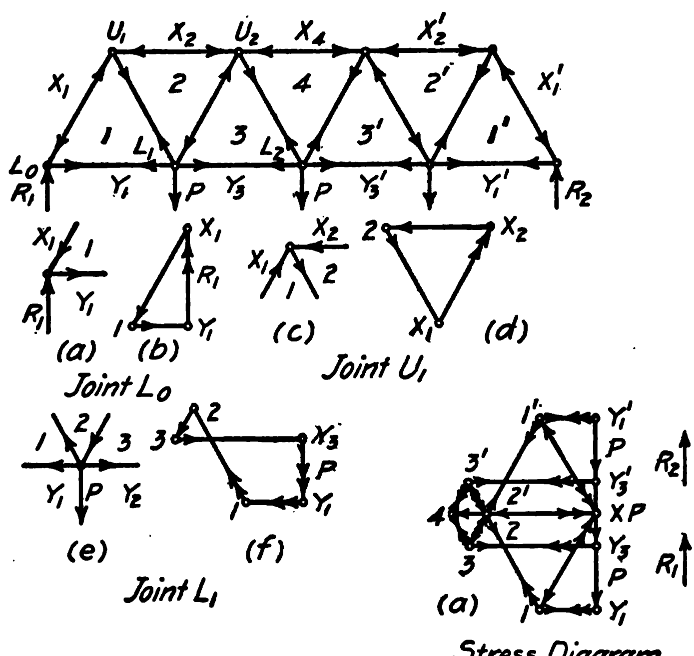



FIG. 9.

the stress diagram and truss is indicated by arrows acting toward the ends of the stress lines and toward the joints, respectively, and tension is indicated by arrows acting away from the

ends of the stress lines and away from the joints, respectively. The first time a stress is used a single arrow, and the second time the stress is used a double arrow is used to indicate direction. It will be seen that the upper chords are in compression, while the lower chord is in tension. The stress diagram in (k) Fig. 8 is called a "Maxwell diagram" or a "reciprocal polygon diagram."

The notation used is known as Bow's notation, in which points in the truss diagram become areas in the stress diagram, and areas in the truss diagram become points in the stress diagram. The method of graphic resolution is the method most commonly used for calculating stresses in roof trusses and simple framed structures with inclined chords.

For the analysis of the stresses in roof trusses, see the author's book, "The Design of Steel Mill Buildings."

*Warren Bridge Truss.*—In Fig. 9 the dead load stresses in a Warren bridge truss loaded on the lower chord, are calculated by the method of graphic resolution. In the stress diagram the loads are laid off from the bottom upwards. The details of the solution can easily be followed by reference to Fig. 9 and Fig. 8. It will be seen that the upper chord of the truss is in compression, while the lower chord is in tension.

**MOMENTS.**—In calculating the stresses in a truss by moments, the fundamental equation for equilibrium for rotation

$$\Sigma \text{ moments of forces about any point} = 0 \qquad (3)$$

is applied to parts of the structure. Equation (3) may be solved either by algebra or by graphics. Before applying equation (3) to the parts of a structure it will be necessary to discuss a few fundamental principles.

**Equilibrium of Non-concurrent Forces.**—If the forces are non-concurrent (do not all meet in a common point), the condition that the force polygon close is a necessary, but not a sufficient condition for equilibrium. For example, take the three equal forces $P_1$, $P_2$ and $P_3$, making an angle of 120° with each other as in (a) Fig. 10.

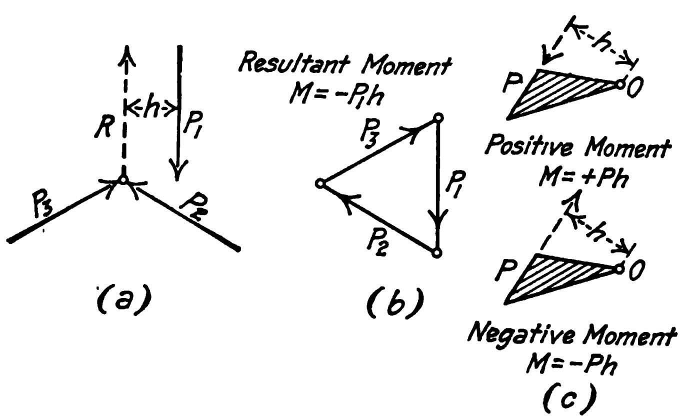



FIG. 10.

The force polygon (b) closes, but the system is not in equilibrium. The resultant, $R$, of $P_2$ and $P_3$ acts through their intersection and is parallel to $P_1$, but is opposite in direction. The system of forces is in equilibrium for translation, but is not in equilibrium for rotation.

The resultant of this system is a couple with a moment $= -P_1 \cdot h$, moments clockwise being considered negative and counter-clockwise positive, (c) Fig. 10. The equilibrant of the system in (a) Fig. 10 is a couple with a moment $= +P_1 \cdot h$.

*A couple.*—A couple consists of two parallel forces equal in amount, but opposite in direction. The arm of the couple is the perpendicular distance between the forces. The moment of a couple is equal to one of the forces multiplied by the arm. The moment of a couple is constant about any point in the plane and may be represented graphically by twice the area of the triangle having one of the forces as a base and the arm of the couple as an altitude. The moment of a force about any point may be represented graphically by twice the area of a triangle, as shown in (c) Fig. 10.

It will be seen from the preceding discussion, that in order that a system of non-concurrent forces be in equilibrium it is necessary that the resultant of all forces save one shall coincide with the one and be opposite in direction. Three non-concurrent forces can not be in equilibrium unless they are parallel. The resultant of a system of non-concurrent forces may be a single force or a couple.

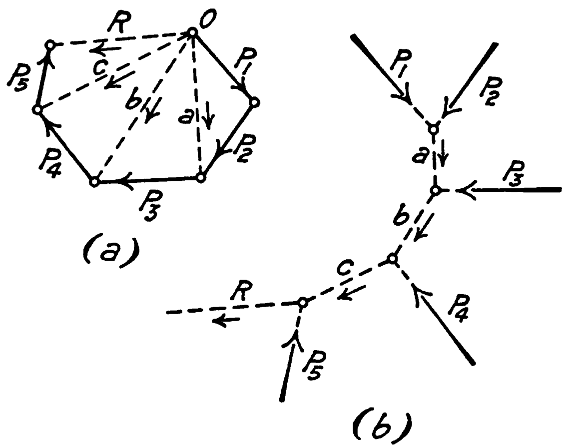



FIG. 11.

**Equilibrium Polygon.** *First Method.*—In Fig. 11 the resultant, $a$, of $P_1$ and $P_2$ acts through their intersection and is equal and parallel to $a$ in the force polygon (a); the resultant, $b$, of $a$ and $P_3$ acts through their intersection and is equal and parallel to $b$ in the force polygon; the resultant, $c$, of $b$ and $P_4$ acts through their intersection and is equal and parallel to $c$ in the force polygon; and finally the resultant, $R$, of $c$ and $P_5$ acts through their intersection and is equal and parallel to $R$ in the force polygon. $R$ is therefore the resultant of the entire system of forces. If $R$ is replaced by an equal and opposite force, $E$, the system of forces will be in equilibrium. Polygon (a) in Fig. 11 is called a force polygon and (b) is called a "funicular" or an "equilibrium" polygon. It will be seen that the magnitude and direction of the resultant of a system of forces is given by the closing line of the force polygon, and the line of action is given by the equilibrium polygon.

The force polygon in (a) Fig. 12 closes and the resultant, $R$, of the forces $P_1$, $P_2$, $P_3$, $P_4$, $P_5$ is parallel and equal to $P_6$, and is opposite in direction. The system is in equilibrium for translation, but is not in equilibrium for rotation. The resultant is a couple with a moment $= -P_6 \cdot h$. The equilibrant of the system of forces will be a couple with a moment $= +P_6 \cdot h$. From the preceding discussion it will be seen that if the force polygon for any system of non-concurrent forces closes the resultant will be a couple. If there is perfect equilibrium the arm of the couple will be zero.

*Second Method.*—Where the forces do not intersect within the limits of the drawing board, or where the forces are parallel, it is not possible to draw the equilibrium polygon as shown in Fig. 11 and Fig. 12, and the following method is used:

The point *o*, (a) Fig. 13, which is called the pole of the force polygon, is selected so that the strings *a–o*, *b–o*, *c–o*, *d–o* and *e–o* in the equilibrium polygon (b), which are drawn parallel to the corresponding rays in the force polygon (a), will make good intersections with the forces which they replace or equilibrate.

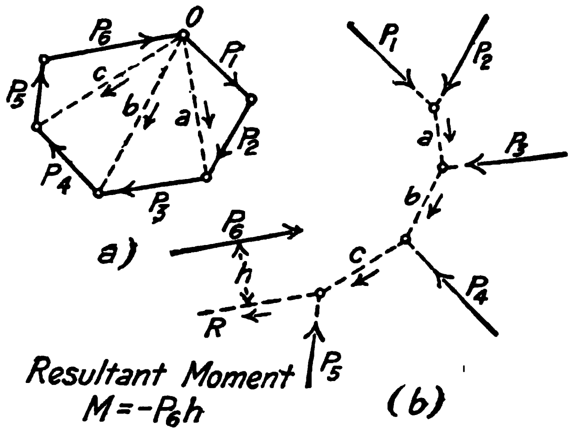



FIG. 12.

In the force polygon (a), $P_1$ is equilibrated by the imaginary forces represented by the rays *o–a* and *b–o*, acting as indicated by the arrows within the triangle; $P_2$ is equilibrated by the imaginary forces represented by the rays *o–b* and *c–o*, acting as indicated by the arrows within the triangle; $P_3$ is equilibrated by the imaginary forces represented by the rays *o–c* and *d–o*, acting as indicated by the arrows within the triangle; and $P_4$ is equilibrated by the imaginary forces *o–d* and *e–o*, acting as indicated by the arrows within the triangle. The imaginary forces are all neutralized except *a–o* and *o–e*, which are seen to be components of the resultant, *R*.

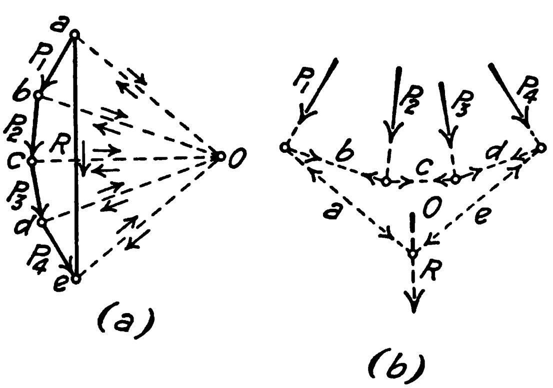



FIG. 13.

To construct the equilibrium polygon, take any point on the line of action of $P_1$ and draw strings *o–a* and *o–b* parallel to rays *o–a* and *o–b*, *b–o* is the equilibrant of *o–a* and $P_1$; through the intersection of string *o–b* and $P_2$ draw string *c–o* parallel to ray *c–o*, *c–o* is the equilibrant of *o–b* and $P_2$; through the intersection of string *c–o* and $P_3$ draw string *d–o* parallel to ray *d–o*, *d–o* is the equilibrant of *c–o* and $P_3$; and through the intersection of string *d–o* and $P_4$ draw string *e–o*

parallel to ray $e$–$o$, $e$–$o$ is the equilibrant of $d$–$o$ and $P_4$. Strings $o$–$a$ and $e$–$o$ acting as shown are components of the resultant, $R$, which will be parallel to $R$ in the force polygon and acts through the intersections of strings $o$–$a$ and $e$–$o$.

The imaginary forces represented by the rays in the force polygon may be considered as components of the forces and the analysis made on that assumption with equal ease.

It is immaterial in what order the forces are taken in drawing the force polygon, as long as the forces all act in the same direction around the force polygon, and the strings meeting on the lines of the forces in the equilibrium polygon are parallel to the rays drawn to the ends of the same forces in the force polygon.

The imaginary forces $a$–$o$, $b$–$o$, $c$–$o$, $d$–$o$, $e$–$o$ are represented in magnitude and in direction by the rays of the force polygon to the same scale as the forces $P_1$, $P_2$, $P_3$, $P_4$. The strings of the equilibrium polygon represent the imaginary forces in line of action and direction, but not in magnitude.

**Graphic Moments.**—In Fig. 14 (b) is a force polygon and (a) is an equilibrium polygon for the system of forces $P_1$, $P_2$, $P_3$, $P_4$. Draw the line $M$–$N = y$, parallel to the resultant, $R$, and with ends on strings $o$–$e$ and $o$–$a$ produced. Let $r$ equal the altitude of the triangle $L$–$M$–$N$, and $H$ equal the altitude of the similar triangle $o$–$e$–$a$. $H$ is the pole distance of the resultant, $R$.

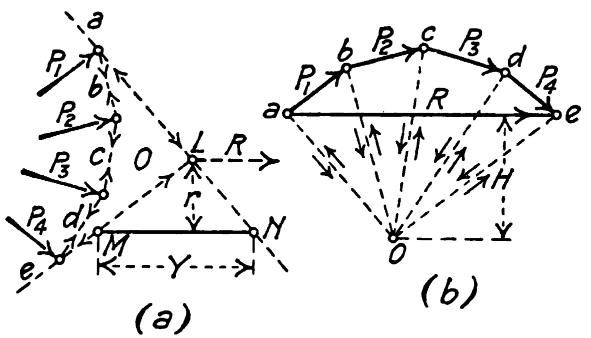



FIG. 14.

Now, in the similar triangles $L$–$M$–$N$ and $o$–$e$–$a$

$$R : y :: H : r$$

and

$$R \cdot r = H \cdot y$$

But $R \cdot r = M$ = moment of resultant $R$ about any point in the line $M$–$N$ and therefore

$$M = H \cdot y \tag{12}$$

The statement of the principle just demonstrated is as follows:

*The moment of any system of coplanar forces about any point in the plane is equal to the intercept on a line drawn through the center of moments and parallel to the resultant of all the forces, cut off by the strings which meet on the resultant, multiplied by the pole distance of the resultant.* It should be noted that in all cases the intercept is a distance and the pole distance is a force.

This property of the equilibrium polygon is frequently used in calculating the bending moments in beams and trusses which are loaded with vertical loads.

**Bending Moments in a Beam.**—It is required to find the moment at the point $M$ in the simple beam loaded as in (b) Fig. 15. The moment at $M$ will be the algebraic sum of the moments

of the forces to the left of $M$. The moment of $P_1 = H \times B\text{–}C$, the moment of $P_2 = H \times C\text{–}D$ and the moment of $R_1 = -H \times B\text{–}A$. The moment at $M$ will therefore be

$$M_1 = H \times B\text{–}C + H \times C\text{–}D - H \times B\text{–}A = -H \times A\text{–}D = -H \cdot y$$

The moment of the forces to the right of $M$ may in like manner be shown to be

$$M_1 = +H \cdot y$$

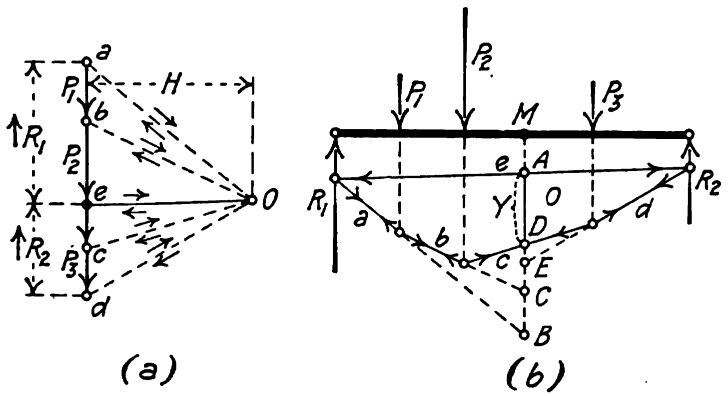



FIG. 15.

In like manner the bending moment at any point in the beam may be shown to be the ordinate of the equilibrium polygon multiplied by the pole distance. The ordinate is a distance and is measured by the same scale as the beam, while the pole distance is a force and is measured by the same scale as the loads.

**Equilibrium Polygon as a Framed Structure.**—In (a) Fig. 16 the rigid triangle supports the load $P_1$. Construct a force polygon by drawing rays $a$–1 and $c$–1 in (b) parallel to sides $a$–1 and

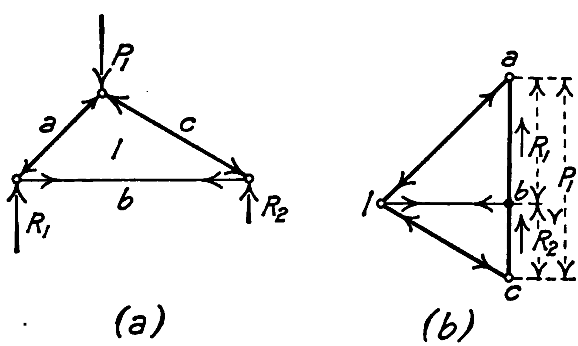



FIG. 16.

$c$–1, respectively, in (a), and through pole 1 draw 1–$b$ parallel to side 1–$b$ in (a). The reactions $R_1$ and $R_2$ will be given by the force polygon (b), and the rays 1–$a$, 1–$c$ and 1–$b$ represent the stresses in the members 1–$a$, 1–$c$ and 1–$b$, respectively, in the triangular structure. The stresses in 1–$a$ and 1–$c$ are compression and the stress in 1–$b$ is tension, forces acting toward the joint indicating compression and forces acting away from the joint indicating tension. Triangle (a) is therefore an equilibrium polygon, and polygon (b) is a force polygon for the force $P_1$.

From the preceding discussion it will be seen that the internal stresses at any point or in any section hold in equilibrium the external forces meeting at the point, or on either side of the section.

**Algebraic Moments. Stresses in a Roof Truss.**—The reactions may be found by applying the fundamental equations of equilibrium to the structure as a whole. In the truss in (a) Fig. 17 by taking moments about the right reaction we have

$$R_1 \times 6d = 5P_1 \times 3d$$

$$R_1 = \tfrac{5}{2}P_1 = R_2$$

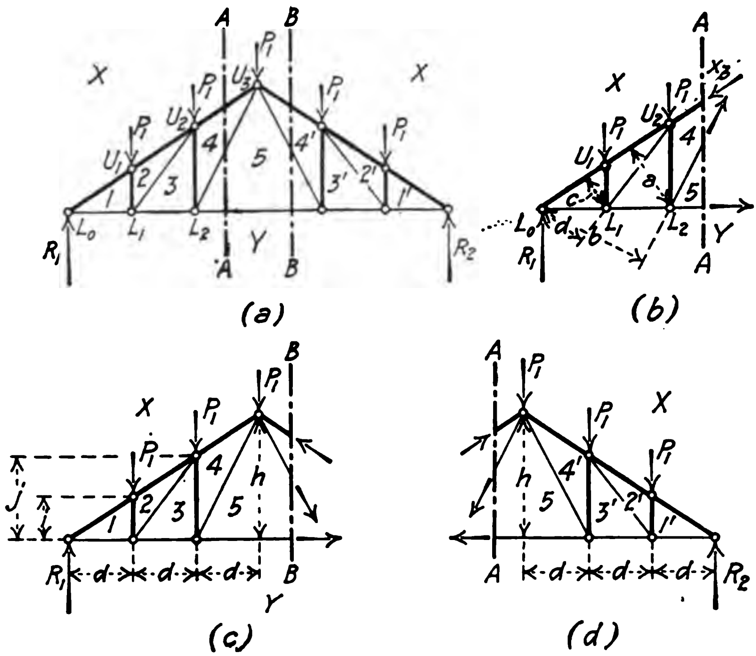



FIG. 17.

To find the stresses in the members of the truss in (a) Fig. 17, proceed as follows: Cut the truss by means of plane $A$–$A$, as in (b), and replace the stresses in the members cut away with external forces. These forces are equal to the stresses in the members in amount, but opposite in direction, and produce equilibrium.

To obtain stress 4–$x$ take center of moments at $L_2$, and take moments of external forces

$$4\text{–}x \times a + P_1 \times d - R_1 \times 2d = 0$$

$$4\text{–}x = \frac{R_1 \times 2d - P_1 \cdot d}{a} = \frac{4P \cdot d}{a} \text{ (compression)}$$

To obtain stress in 4–5 take center of moments at $L_0$, and take moments of external forces

$$4\text{–}5 \times b - 2P_1 \times \tfrac{3}{2}d = 0$$

$$4\text{–}5 = \frac{3P_1 \cdot d}{b} \text{ (tension)}$$

To obtain the stress in 5–$y$ take center of moments at joint $U_2$ in (c), and take moments of external forces

$$5\text{–}y \times h - R_1 \times 3d + 3P_1 \cdot d = 0$$

$$5\text{–}y = \frac{3R_1 \cdot d - 3P_1 \cdot d}{h} = \frac{9P_1 \cdot d}{2h} \text{ (tension)}$$

*To Determine Kind of Stress.*—If the unknown external force is always taken as acting from the outside toward the cut section, *i. e.*, is always assumed to cause compression, the sign of the result will indicate the kind of stress. A plus sign will indicate that the assumed direction was correct and that the stress is compression, while a minus sign will indicate that the assumed direction was incorrect and that the stress is tension.

In calculating stresses by algebraic moments, therefore, always observe the following rule:

***Assume the unknown external force as acting from the outside toward the cut section; a plus sign for the result will then show that the stress in the member is compression, and a minus sign will indicate that the stress in the member is tension.***

The stresses in the web members 3–4, 2–3, 1–2, are found by taking moments about joint $L_0$ as a center. The stresses in $y$–3 and $y$–1 are found by taking moments about joints $U_2$ and $U_1$, respectively; and the stresses in $x$–2 and $x$–1 are found by taking moments about joint $L_1$.

The method of algebraic moments is the most common method used for calculating the stresses in bridge trusses with inclined chords, and similar frameworks which carry moving loads.

**Stresses in a Bridge Truss.**—Calculate reaction $R_1$ by taking moments of the vertical forces about joint $L_0'$. Then $R_1 \times L = 6P \cdot L/2$, and $R_1 = 3P = R_2$. To calculate the stress in any

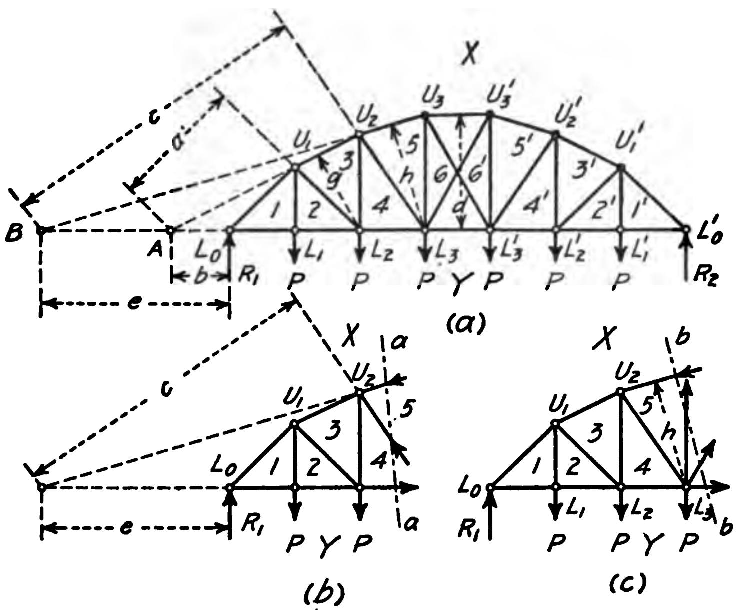



FIG. 18.

member in the truss, pass a section cutting the member in which the stress is required, and cutting away the truss on one side of the section. The stresses in the members cut away are assumed as replaced by external forces acting in the line of the member and equal to the stresses in amount.

To calculate the stresses take the center of moments so that there will be but one unknown stress. The solution of the equation of moments about this center of moments will give the required stress. To calculate the stress in 4–5 in (b) Fig. 18, pass the section *a–a*, cutting away the right side of the truss, and take the center of moments at the intersection of the top and bottom chords. Now 5–*x* and 4–*y* act through the center of moments and produce no moment. The moment of the stress in 4–5 acting from the outside toward the cut section with an arm *c*, holds in equilibrium the reaction $R_1$, and the two loads, *P*. The sign of the result will determine the kind of stress, minus for tension and plus for compression. To calculate the stress in the top chord $U_2U_3$, pass section *b–b* in (c) and take moments about joint $L_2$.

**Graphic Moments.**—The bending moment at any point in a truss may be found by means of a force and equilibrium polygon as in (b) and (a) Fig. 19. To determine the stress in 4–*x*, cut section *A–A* and take moments about joint $L_2$ as in Fig. 19. The moment of the external

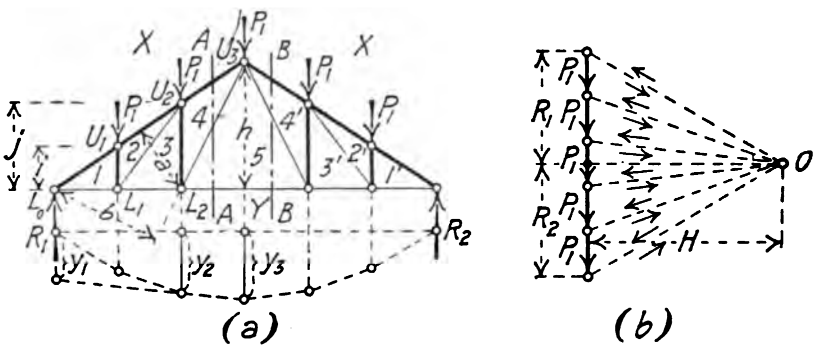



FIG. 19.

forces on the left of $L_2$ will be $M_2 = -H \cdot y_2$, and stress

$$4\text{–}x = -M_2/a = +H \cdot y_2/a$$

To obtain stress in 4–5 take center of moments at joint $L_0$, and stress

$$4\text{–}5 = M_1/b = -H \cdot y_1/b$$

To obtain stress in 5–*y* take center of moments at joint $U_3$, and stress

$$5\text{–}y = M_3/h = -H \cdot y_3/h$$

The method of graphic moments is principally used to explain other methods and is little used as a direct method of calculation.

# CHAPTER II.

## STRESSES IN BEAMS.

**Introduction.**—Simple and cantilever beams, only, will be considered in this chapter. For the calculation of stresses in continuous beams, see the author's "Steel Mill Buildings," Chapter XVa.

**Reactions of a Simple Beam.**—A force and an equilibrium polygon may be used to obtain the reactions of a beam loaded with a load $P$, as in Fig. 1.

The force polygon (b) is drawn with a pole $O$ at any convenient point, and rays $O$–$a$ and $O$–$c$ are drawn. Now from the fundamental conditions for equilibrium for translation we have

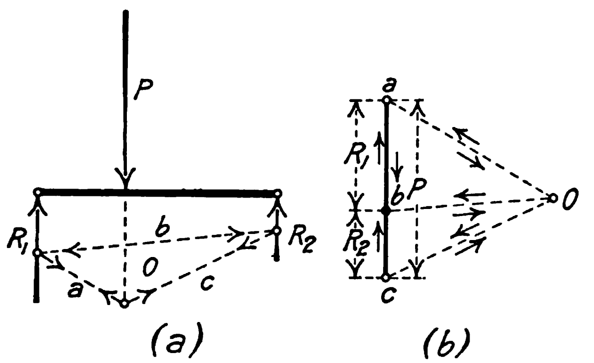



FIG. 1.

$P = R_1 + R_2$. At any convenient point in the line of action of $P$, draw the strings $O$–$a$ and $O$–$c$ parallel to the rays $O$–$a$ and $O$–$c$, respectively, in the force polygon. The imaginary forces $a$–$O$ and $O$–$c$ acting as shown, equilibrate the force $P$. The imaginary force $a$–$O$ acting in a reverse direction, as shown, is an equilibrant of $R_1$, and the imaginary force $c$–$O$, acting in a reverse direction, is an equilibrant of $R_2$. The remaining equilibrant of $R_1$ and of $R_2$ must coincide and be equal in amount, but opposite in direction. The string $b$–$O$ is the remaining equilibrant of $R_1$ and also of $R_2$, and is called the closing line of the equilibrium polygon. The ray $b$–$O$ drawn parallel to the string $b$–$O$ divides $P$ in two parts, which are equal to the reactions $R_1$ and $R_2$.

**Reactions of a Cantilever Beam.**—As a second example let it be required to find the reactions of the overhanging beam shown in Fig. 2.

Construct a force polygon with pole $O$, as in (b), and draw an equilibrium polygon, as in (a). The ray $O$–$d$, drawn parallel to the closing line $O$–$d$ in (a), determines the reactions. In this case reaction $R_1$ is negative. It should be noted that the closing line in an equilibrium polygon must have its ends on the two reactions.

The ordinate to the equilibrium polygon at any point, multiplied by the pole distance, $H$, will give the bending moment in the beam at a point immediately above it.

**Moments and Shears in Beams: Concentrated Loads.**—The bending moments in the beam in Fig. 3 may be found by constructing the force polygon (a) and the equilibrium polygon (b) as shown.

The bending moment at any point is then equal to the ordinate to the equilibrium polygon at that point, multiplied by the pole distance, $H$. The ordinate is to be measured to the same scale as the beam, and the pole distance, $H$, is to be measured to the same scale as the loads in the force polygon. The ordinate is a distance and the pole distance is a force.

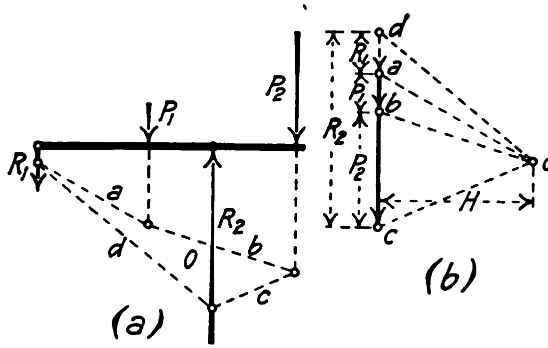



FIG. 2.

Or, if the scale to which the beam is laid off be multiplied by the pole distance measured to the scale of the loads, and this scale be used in measuring the ordinates, the ordinates will be equal to the bending moments at the corresponding points. This is the same as making the pole distance equal to unity. Diagram (b) is called a moment diagram.

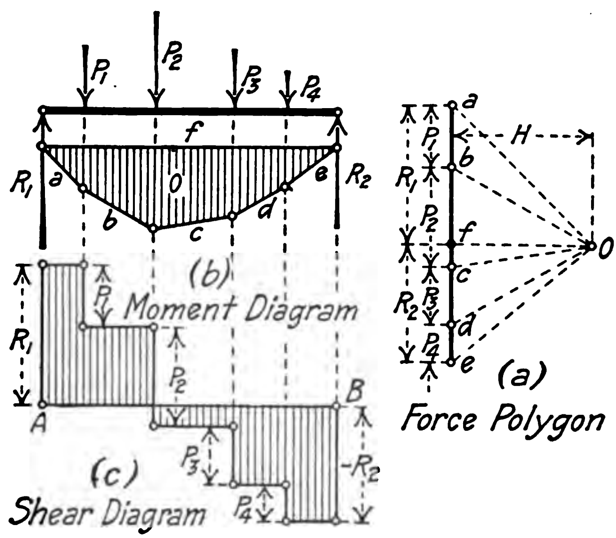



FIG. 3.

Between the left support and the first load the shear is equal to $R_1$; between the loads $P_1$ and $P_2$ the shear equals $R_1 - P_1$; between the loads $P_2$ and $P_3$ the shear equals $R_1 - P_1 - P_2$; between the loads $P_3$ and $P_4$ the shear equals $R_1 - P_1 - P_2 - P_3$; and between load $P_4$ and the right reaction the shear equals $R_1 - P_1 - P_2 - P_3 - P_4 = -R_2$. At load $P_2$ the shear changes

from positive to negative. Diagram (c) is called a shear diagram. It will be seen that the maximum ordinate in the moment diagram comes at the point of zero shear.

The bending moment at any point in the beam is equal to the algebraic sum of the shear areas on either side of the point in question. From this we see that the shear areas on each side of $P_2$ must be equal. This property of the shear diagram depends upon the principle that *the bending moment of any point in a simple beam is the definite integral of the shear between either point of support and the point in question.* This will be taken up again in the discussion of beams uniformly loaded, which will now be considered.

**Moments and Shears in Beams: Uniform Loads.**—In the beam loaded with a uniform load of $w$ lb. per lineal foot shown in Fig. 4, the reaction $R_1 = R_2 = \frac{1}{2}w \cdot L$. At a distance $x$ from the left support, the bending moment is

$$M = R_1 \cdot x - w \cdot x^2/2 = \tfrac{1}{2} \times w(L \cdot x - x^2) \qquad (1)$$

which is the equation of the common parabola.

The parabola may be constructed by means of the force and equilibrium polygons, by assuming that the uniform load is concentrated at points in the beam, as is assumed in a bridge truss, and then drawing the force and equilibrium polygons in the usual way, as in Fig. 4. The greater

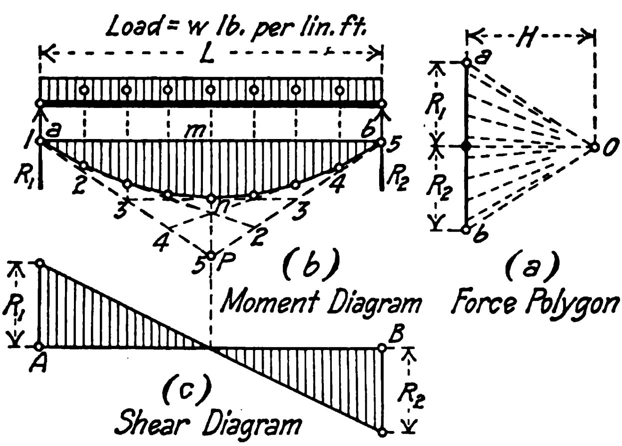



FIG. 4.

the number of segments into which the uniform load is divided, the more nearly will the equilibrium polygon approach the bending moment parabola.

The parabola may be constructed without drawing the force and equilibrium polygons as follows: Lay off ordinate $m-n = n-p$ = bending moment at center of beam $= \frac{1}{8}w \cdot L^2$. Divide $a-p$ and $b-p$ into the same number of equal parts and number them as shown in (b). Join the points with like numbers by lines, which will be tangents to the required parabola. It will be seen in Fig. 4 that points on the parabola are also obtained.

The shear at any point $x$ will be

$$S = R_1 - w \cdot x = \tfrac{1}{2}w \cdot L - w \cdot x = w(L/2 - x) \qquad (2)$$

which is the equation of the inclined line shown in (c) Fig. 4. The shear at any point is therefore represented by the ordinate to the shear diagram at the given point.

*Property of the Shear Diagram.*—Integrating the equation for shear between the limits, $x = 0$ and $x = x$, we have

$$\int_0^x S\cdot dx = \int_0^x w(L/2 - x)dx = \tfrac{1}{2}w(L\cdot x - x^2) \tag{3}$$

which is the equation for the bending moment at any point, $x$, in the beam, and is also the area of the shear diagram between the limits given. From this we see that the bending moment at any point in a simple beam uniformly loaded, is equal to the area of the shear diagram to the left of the point in question. The bending moment is also equal to the algebraic sum of the shear areas on either side of the point.

**Beam With Partial Uniform Load.**—The beam in Fig. 5 is loaded with a load $w$ extending over a length $b$. The bending moments between the left end of the uniform load and the left reaction is $R_1\cdot x$, represented by the ordinates to the straight line $A$–1 in (a); the bending moments in that part of the beam covered by the uniform load is represented by ordinates to the curved line 1–2; while the bending moments to the right of the uniform load are represented by ordinates to the straight line 2–$B$. The ordinates from the straight line 1–2 to the curve 1–2 are the same as for a simple beam with a span $b$ loaded with a uniform load $w$. The shear diagram is shown in ($b$). It will be seen that the maximum bending moment comes at the point of zero shear.

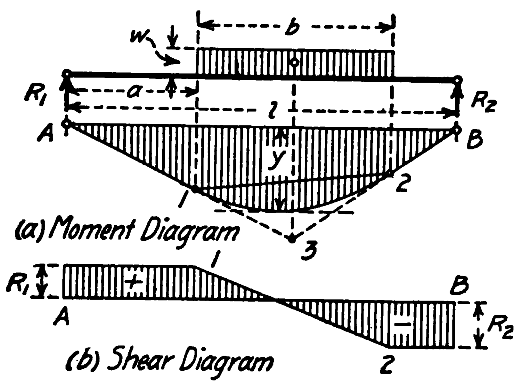



FIG. 5.

**Uniform Moving Loads.**—Let the beam in Fig. 6 be loaded with a uniform load of $p$ lb. per lineal foot, which can be moved on or off the beam.

To find the position of the moving load that will produce a maximum moment at a point a distance $a$ from the left support, proceed as follows: Let the end of the uniform load be at a distance $x$ from the left reaction. Then taking moments about $R_2$ we have

$$R_1 = \frac{(l - x)^2}{2l}p \tag{4}$$

and the moment at the point whose absicssa is $a$ will be

$$M = R_1\cdot a - \frac{(a - x)^2}{2}p = \frac{(l - x)^2}{2l}a\cdot p - \frac{(a - x)^2}{2}p \tag{5}$$

Differentiating (5) with respect to $x$, and placing the derivative of $M$ equal to zero, we have after solving

$$x = 0 \tag{6}$$

Therefore the maximum moment at any point in a beam will occur when the beam is fully loaded.

The bending moment diagram for a beam loaded with a uniform moving load is constructed as in Fig. 4.

To find the position of the moving load for maximum shear at any point in a beam loaded with a moving uniform load, proceed as follows: The left reaction when the end of the moving load is at a distance $x$ from the left reaction will be

$$R_1 = \frac{(l - x)^2}{2l} p \tag{4}$$

and the sheat at a point at a distance $a$ from the left reaction will be

$$S = R_1 - (a - x)p = \frac{(l - x)^2}{2l} p - (a - x)p \tag{7}$$

which is the equation of a common parabola.

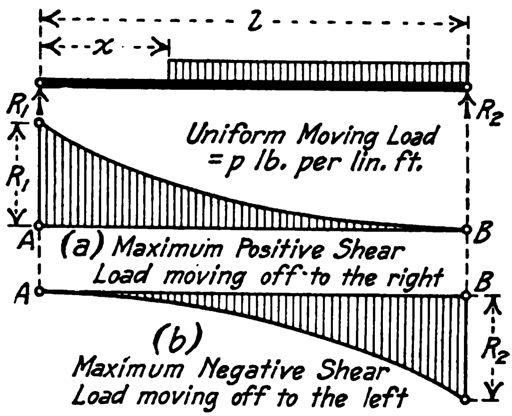



FIG. 6.

By inspection it can be seen that $S$ will be a maximum when $x = a$. The maximum shear at any point in a beam will therefore occur at the end of the uniform moving load, the beam being fully loaded to the right of the point as in (a) Fig. 6 for maximum positive shear, and fully loaded to the left of the point as in (b) Fig. 6 for maximum negative shear.

If the beam is assumed to be a cantilever beam fixed at $A$, and loaded with a stationary uniform load equal to $p$ lb. per lineal foot, and an equilibrium polygon be drawn with a force polygon having a pole distance equal to length of span, $l$, the parabola drawn through the points in the equilibrium polygon will be the maximum positive shear diagram, ($a$) Fig. 6. The ordinate at any point to this shear diagram will represent the maximum positive shear at the point to the same scale as the loads (for the application of this principle to bridge trusses see Fig. 9, Chap. III).

**Concentrated Moving Loads.** *Bending Moments.*—Let a beam be loaded with concentrated moving loads at fixed distances apart as shown in Fig. 7.

To find the position of the loads for maximum moment and the amount of the maximum moment, proceed as follows: The load $P_2$ will be considered first. Let $x$ be the distance of the load $P_2$ from the left support, when the loads produce a maximum moment under load $P_2$.

Taking moments about $R_2$, we have

$$\begin{aligned} R_1 \cdot l &= P_1(l - x + a) + P_2(l - x) + P_3(l - x - b) + P_4(l - x - b - c) \\ &= (l - x)(P_1 + P_2 + P_3 + P_4) + P_1 \cdot a - P_3 \cdot b - P_4(b + c) \end{aligned} \tag{8}$$

and the bending moment under load $P_2$ will be

$$M = R_1 \cdot x - P_1 \cdot a$$
$$= \frac{x(l - x)(P_1 + P_2 + P_3 + P_4) + x[P_1 \cdot a - P_3 \cdot b - P_4(b + c)]}{l} - P_1 \cdot a \qquad (9)$$

Differentiating (9) with respect to $x$, we have

$$\frac{dM}{dx} = \frac{(l - 2x)(P_1 + P_2 + P_3 + P_4) + P_1 \cdot a - P_3 \cdot b - P_4(b + c)}{l} = 0 \qquad (10)$$

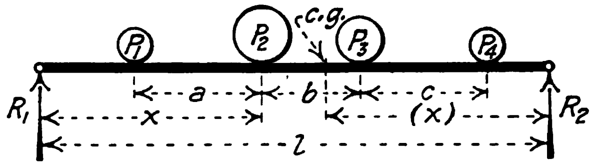



FIG. 7.

and solving (10) for $x$, we have

$$x = \frac{l}{2} + \frac{P_1 \cdot a - P_3 \cdot b - P_4(b + c)}{2(P_1 + P_2 + P_3 + P_4)}. \qquad (11)$$

Now $P_1 \cdot a - P_3 \cdot b - P_4(b + c)$ is the static moment of the loads about $P_2$, and

$$\frac{P_1 \cdot a - P_3 \cdot b - P_4(b + c)}{P_1 + P_2 + P_3 + P_4}$$

= distance from $P_2$ to center of the gravity of all the loads.

*Therefore, for a maximum moment under load $P_2$, it ($P_2$) must be as far from one end as the center of gravity of all the loads is from the other end of the beam, Fig. 7.*

The above criterion holds for all the loads on the beam. The only way to find which load produces the greatest maximum is to try each one, however, it is usually possible to determine by inspection which load will produce a maximum bending moment. For example, the maximum moment in the beam in Fig. 7 will certainly come under the heavy load $P_2$. The above proof may be generalized without difficulty, and the criterion above shown to be of general application.

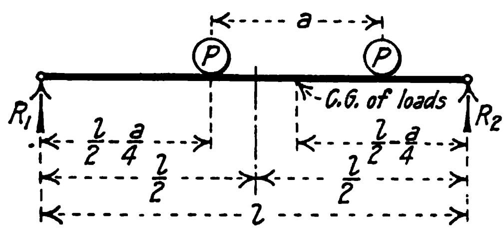



FIG. 8.

For two equal loads, $P = P$, at a fixed distance, $a$, apart as in the case of a traveling crane, Fig. 8, the maximum moment will occur under one of the loads, when

$$x = l/2 - a/4 \qquad (12)$$

Taking moments about the right reaction, we have

$$R_1 \cdot l = P(l - a/2) \qquad (13)$$

and the maximum bending moment is

$$M = R_1(l/2 - a/4)$$
$$= \frac{P(l - a/2)^2}{2l} \qquad (14)$$

There will be a maximum moment when either of the loads satisfies the above criterion, the bending moments being equal.

By equating the maximum moment calculated as above to the moment due to a single load at the center of the beam, it will be found that the above criterion holds only, when

$$a < 0.586l \qquad (15)$$

Where two unequal moving loads are at a fixed distance apart, the greater maximum bending moment will always come under the heavier load.

*Shears.*—The maximum end shear at the left support, for a system of concentrated loads on a simple beam, as in Fig. 8, will occur when the left reaction, $R_1$, is a maximum. This will occur when one of the wheels is infinitely near the left abutment (usually said to be over the left abutment). The load which produces maximum end shear can be easily found by trial.

The maximum shear at any point in the beam will occur when one of the loads is over the point. The criterion for determining which load will cause a maximum shear at any point, $x$, in a beam will now be determined.

In Fig. 8, let the total load on the beam, $P_1 + P_2 + P_3 + P_4 = W$, and let $x$ be the distance from the left support to the point at which we wish to determine the maximum shear.

When load $P_1$ is at the point, the shear will be equal to the left reaction, which is found by substituting $x + a$ for $x$ in (8) to be

$$S_1 = R_1 = \frac{(l - x - a)W + P_1 \cdot a - P_3 \cdot b - P_4(b + c)}{l}$$

and when $P_2$ is at the point, the shear will be

$$S_2 = \frac{(l - x)W + P_1 \cdot a - P_3 \cdot b - P_4(b + c)}{l} - P_1$$

Subtracting $S_2$ from $S_1$, we have

$$S_1 - S_2 = \frac{P_1 \cdot l - W \cdot a}{l}$$

Now $S_1$ will be greater than $S_2$ if $P_1 \cdot l$ is greater than $W \cdot a$, or if

$$P_1/a > W/l \qquad (16)$$

The criterion for maximum shear at any point, therefore, is as follows:

*The maximum positive shear in any section of a beam occurs when the foremost load is at the section, provided $W/l$ is not greater than $P_1/a$. If $W/l$ is greater than $P_1/a$, the greatest shear will occur when some succeeding load is at the point.*

Having determined the position of the moving loads for maximum moment and maximum shear, the amount of the moment and shear can be obtained as in the case of beams loaded with stationary loads.

**DESIGN OF BEAMS.**—Having calculated the maximum bending moments and shears in the beam the stresses are calculated as follows:

**Shearing Stresses.**—The shear is assumed as uniformly distributed over the cross-section of the beam, and the shearing stress will be equal to the shear $S_1$ divided by the area of the beam. The actual shearing stress in the beam must be less than the allowable shearing stress.

**Tensile and Compression Stresses.**—A simple beam carried on two end supports will have its upper fibers in compression and its lower fibers in tension, there being no stress on the neutral axis of the beam.

The stress due to bending moment will be given by the formula

$$S = M \cdot c/I \tag{17}$$

where $S$ is the unit stress on the extreme fiber, being tension on the convex side and compression on the concave side of the beam, $M$ is the bending moment of the forces on one side of the given section, $c$ is the distance in inches from the neutral axis of the beam to the extreme fiber considered, and $I$ is the moment of inertia of the cross-section of the beam in inches to the fourth power.

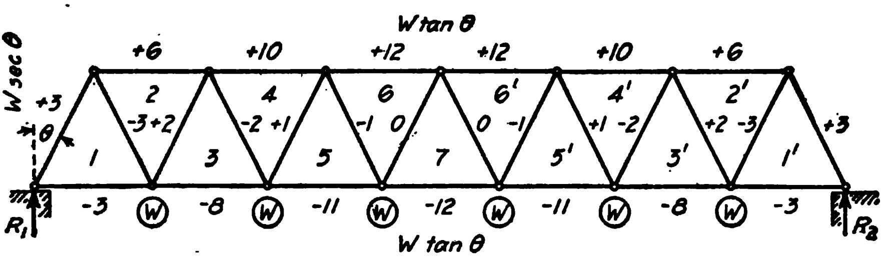
The allowable unit stresses for shear, tension and compression are given in Chapter IX and in Appendix I.

# CHAPTER III.

## STRESSES IN HIGHWAY BRIDGE TRUSSES.

**LOADS.**—The loads on highway bridges are commonly specified as a certain number of pounds per square foot of floor surface, or per lineal foot of truss or bridge. The live load is assumed as applied at the panel points of the loaded chord, while the dead load may be assumed as all applied on the loaded chord, or assumed as partly applied on the loaded chord and partly on the unloaded chord (usually two-thirds on the loaded chord and one-third on the unloaded chord). In this discussion the dead load will be assumed as applied at the panel points in the loaded chord. Equal panel lengths and joint loads will also be assumed. For extracts from standard specifications for dead loads of highway bridges, see Chapter IX.

**Algebraic Resolution.***—Let the Warren truss, in Fig. 1, have dead loads applied at the joints of the lower chord as shown. From the fundamental equations for equilibrium for rotation and translation, reaction $R_1 = R_2 = 3W$.




*Dead Load Coefficients*

FIG. 1.

The stresses in the members are calculated as follows: Resolving at the left reaction, stress in $1\text{–}x = +3W\cdot\sec\theta$, and stress in $1\text{–}y = -3W\cdot\tan\theta$. Resolving at first joint in upper chord, stress in $1\text{–}2 = -3W\cdot\sec\theta$, and stress in $2\text{–}x = +6W\cdot\tan\theta$. Resolving at second joint in lower chord, stress $2\text{–}3 = +2W\cdot\sec\theta$, and stress $3\text{–}y = -8W\cdot\tan\theta$. And in like manner the stresses in the remaining members are found as shown. The coefficients shown in Fig. 1 for the chords are to be multiplied by $W\cdot\tan\theta$; while those for the webs are to be multiplied by $W\cdot\sec\theta$.

It will be seen that the coefficients for the web stresses are equal to the shears in the respective panels. Having found the shears in the different panels of the truss, the remaining coefficients may be found by resolution. Pass a section through any panel and the algebraic sum of the coefficients will be equal to zero. Therefore, if two coefficients are known, the third will be equal to the algebraic sum of the two, with sign changed.

Beginning with coefficient of member $1\text{–}y$, which is known and equals $-3$;

$$\text{coefficient of } 2\text{–}x = -(-3-3) = +6;$$

$$\text{coefficient of } 3\text{–}y = -(+6+2) = -8;$$

$$\text{coefficient of } 4\text{–}x = -(-8-2) = +10;$$

* Also called "Method of Sections."

coefficient of $5-y = -(+10+1) = -11$;
coefficient of $6-x = -(-11-1) = +12$;
coefficient of $7-y = -(+12+0) = -12$.

*Loading for Maximum Stresses.*—The effect of different positions of the loads on a Warren truss will now be investigated.

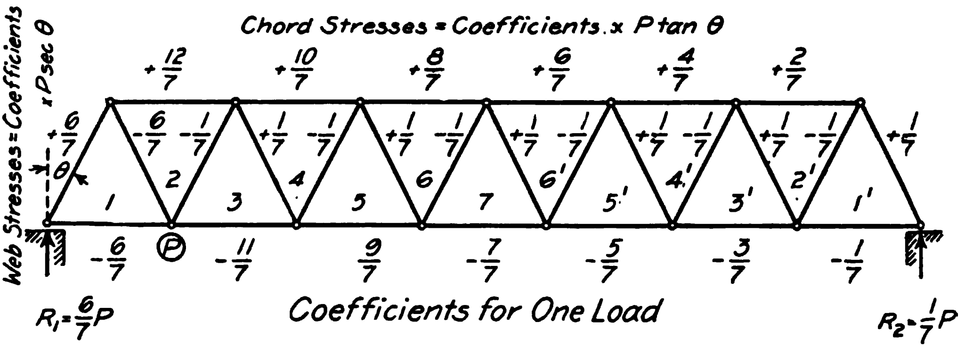



FIG. 2.

Let the truss in Fig. 2 be loaded with a single load $P$ as shown. The left reaction, $R_1 = \frac{6}{7}P$, and the right reaction, $R_2 = P/7$. The stress in $1-y = -\frac{6}{7}P \cdot \tan\theta$, and stress in $1-x = +\frac{6}{7}P \cdot \sec\theta$. The stress in $1-2 = -\frac{6}{7}P \cdot \sec\theta$, and stress in $2-3 = -\frac{1}{7}P \cdot \sec\theta$, etc. The remaining coefficients are found as in the case of dead loads by adding coefficients algebraically and changing the sign of the result.

In Fig. 3 the coefficients for a load applied at each joint in turn are shown for the different members; the coefficients for the load on left being given in the top line.

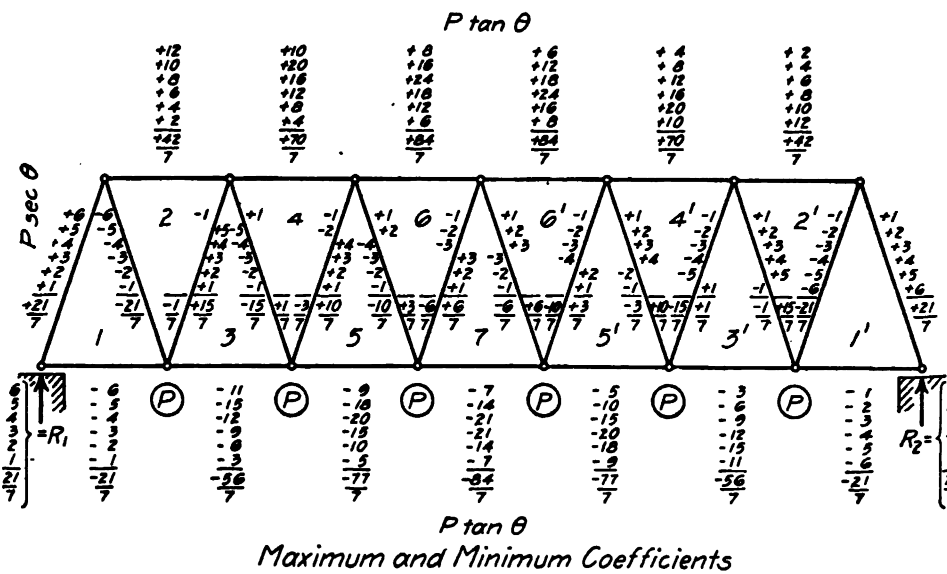



FIG. 3.

The following conclusions may be drawn from Fig. 3:

1. All loads produce compressive stresses in the top chord and tensile stresses in the bottom chord.

2. All the loads on one side of a panel produce the same kind of stress in the web members that are inclined in the same direction on that side.

3. For maximum stresses in the chords, therefore, the truss should be fully loaded.

4. For maximum stresses in the web members the longer segment into which the panel divides the truss should be fully loaded; while for minimum stresses in the web members the shorter segment of the truss should be fully loaded.

The conditions for maximum loading of a truss with equal joint loads are therefore seen to be essentially the same as the maximum loading of a beam with a uniform live load.

For a discussion of the conditions of loading for maximum and minimum stresses in trusses by means of Influence Diagrams, see Chapter IV.

**Stresses in a Warren Truss.**—The coefficients for the maximum and minimum stresses in a Warren truss, due to live load are shown in Fig. 4.

These coefficients are seen to be the algebraic sum of the coefficients for the individual loads given in Fig. 3. The live load chord coefficients are the same as for dead load, and if found directly are found in the same manner.

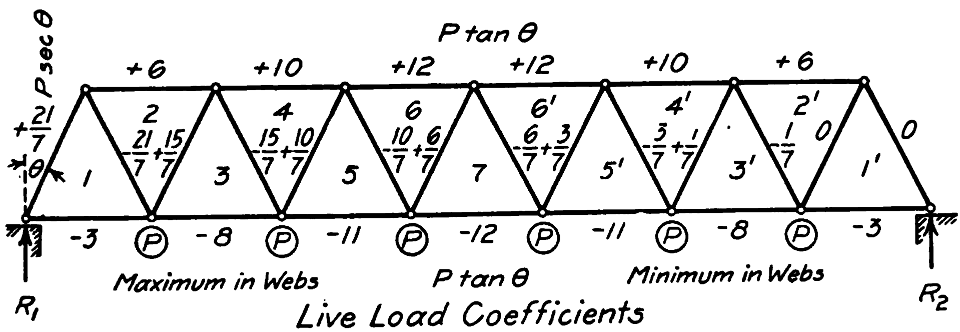



FIG. 4.

The maximum web coefficients may be found directly by taking off one load at a time, beginning at the left. The left reaction, which may be found by algebraic moments, will in each case be the coefficient of the maximum stress in the panel to the left of the first load. A rule for finding the coefficient of left reaction for any loading is as follows: *Multiply the number of loads on the truss by one-half the number of loads plus unity, and divide the product by the number of panels in the truss, the result will be the coefficient of the left reaction.*

If the second differences of the maximum coefficients in the web members are calculated, they will be found to be constant, which shows that the coefficients are equal to the ordinates of a parabola.

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Coefficients, | 21 | | 15 | | 10 | | 6 | | 3 | | 1 |
| 1st differences, | | 6 | | 5 | | 4 | | 3 | | 2 | |
| 2nd differences, | | | 1 | | 1 | | 1 | | 1 | | |

SECOND DIFFERENCES OF NUMERATORS OF WEB COEFFICIENTS.

This relation gives an easy method for checking up the maximum web coefficients, since the numerators of the coefficients are always the same beginning with zero in the first panel on the right and progressing in order 1, 3, 6, 10, etc.; the denominators always being the number of panels in the truss.

It will also be found that the second differences of the upper or lower chord coefficients are constant, showing that the chord stresses are proportional to the ordinates to a parabola.

It should be noted that in the Warren truss the web members meeting on the unloaded chord always have stresses equal in amount, but opposite in sign.

The web member 6–7 has a zero dead load stress, and a complete reversal due to live load, making it necessary to design the member to take both tension and compression.

For the calculation of the maximum and minimum stresses in a Warren truss, see Problem 10, Chapter VII.

**Stresses in a Pratt Truss.**—In the Pratt truss the diagonal members are tension members, and counters (see dotted members in (c) Fig. 5 )must be supplied where there is a reversal of stress. The coefficients for the dead and live load stresses in the Pratt truss, shown in (a) and (b) Fig. 5,

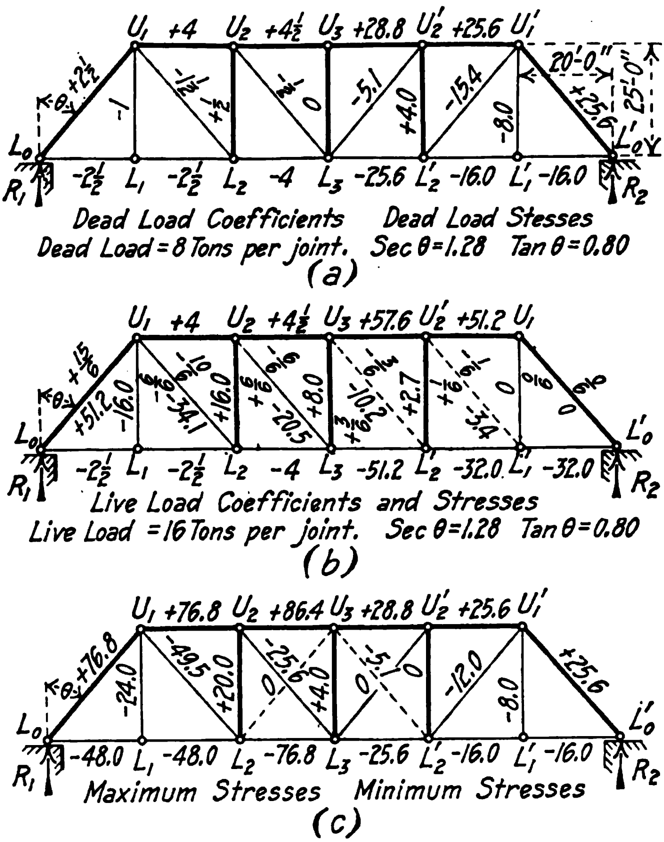



FIG. 5.

are found in the same manner as for a Warren truss. The member $U_1L_1$ acts as a hanger and carries only the load at its lower end. The stresses in the chords are found by multiplying the coefficients by $W \cdot \tan\theta$, and in the inclined webs by multiplying the coefficients by $W \cdot \sec\theta$. The stresses in the posts are equal to the vertical components of the stresses in the inclined web members meeting them on the unloaded chord.

The maximum chord stresses shown on the left of (c), are equal to the sum of the live and dead load chord stresses. The minimum chord stresses shown on the right of (c), are equal to the dead load chord stresses.

The maximum and minimum web stresses are found by adding, algebraically, the stresses in the members due to dead and live loads.

Since the diagonal web members in a Pratt truss can take tension only, counters must be supplied as $U_3L_3'$ in panel $L_3'L_3$. The tensile stress in a counter in a panel of a Pratt truss is always equal to the compressive stress that would occur in the main diagonal web member in the panel, if it were possible for it to take compression. Care must always be used to calculate the corresponding stresses in the vertical posts.

For the calculation of the maximum and minimum stresses in a Pratt truss, see Problem 11, Chapter VII.

**Method of Shear Increments.**—The loads on a beam or truss first produce shears, which in turn produce bending stresses in the chords. In (a) Fig. 5 it will be seen that member $U_2L_3$ carries the shear in the panel of $\frac{1}{2}W$, which produces a stress of $-\frac{1}{2}W\cdot\sec\theta$ in the member. The difference in the stresses in $U_1U_2$ and $U_2U_3$ is seen to be the horizontal component of the stress in $U_2L_3$, or the shear increment in the panel. The shear increment may be calculated as follows: The shear in the panel $L_2L_3$ is $W/2$ and may be assumed to act a differential to the right of joint $L_2$. Now take moments about $L_3$, and pass a section cutting $U_2U_3$, $U_2L_3$ and $L_2L_3$ just to the right of $L_2$, and cutting away the truss to the left. Now the shear, $S$, represents the resultant of the vertical forces to the left of the panel. Then for equilibrium the stress in $U_2U_3$ will be equal to the stress in $U_1U_2$ found by taking moments about joint $L_2$, plus the shear increment $I = (S \times l)/d, = \frac{1}{2}W\cdot\tan\theta$, where $l$ = panel length and $d$ = depth of truss.

**GRAPHIC RESOLUTION.**—The stresses in a Warren truss due to dead loads are calculated by graphic resolution in Problem 1, Chapter VII. The solution is the same as for the truss in Fig. 9, Chapter I. The loads, beginning with the first load on the left, are laid off from the bottom upwards. The analysis of the solution is shown on the stress diagram and truss, and needs no explanation.

From the stresses in the members it is seen (a) that web members meeting on the unloaded chord have stresses equal in amount but opposite in sign, and (b) that the lower chord stresses are the arithmetical means of the upper chord stresses on each side.

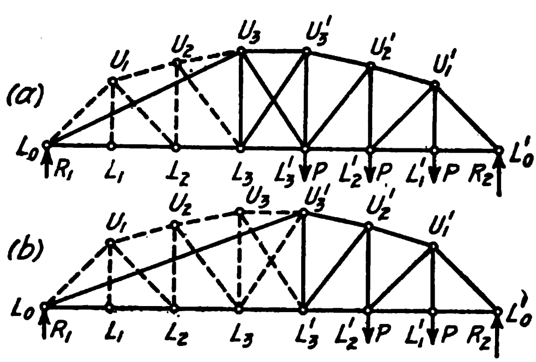



FIG. 6.

The live load chord stresses may be obtained from the dead load stress diagram, by changing the scale, or by multiplying the dead load stresses by a constant.

The live load web stresses may be obtained by calculating the left reactions for the loading that gives a maximum shear in the panel (no loads occurring between the panel and the left

reaction), and then constructing the stress diagram up to the member whose stress is required. In a truss with parallel chords it is only necessary to calculate the stress in the first web member for any given reaction, since the shear is constant between the left reaction and the panel in question.

The live load web stresses may all be obtained from a single diagram as follows: With an assumed left reaction of, say, 100,000 lb. construct a stress diagram on the assumption that the truss is a cantilever fixed at the right abutment, and that there are no loads on the truss. Then the maximum stress in any web member will be equal to the stress scaled from the diagram, divided by 100,000, multiplied by the left reaction that produces the maximum stress. This method is a very convenient one for finding the stresses in a truss with inclined chords. For examples, see Problem 19, and Problem 20, Chapter VII.

In calculating the maximum and minimum stresses in a bridge truss by graphic resolution the labor in constructing the stress diagram may be reduced by replacing the truss to the left of the panel by a triangle as in (a) or (b) in Fig. 6. In (a) the correct stresses will be given in $U_1L_2'$ or $U_2'L_2$, but the correct stress will not be given in $U_2L_2$.

**ALGEBRAIC MOMENTS.**—The dead and live load stresses in a truss with inclined chords are calculated by algebraic moments in Fig. 7. The conditions for maximum loading are the

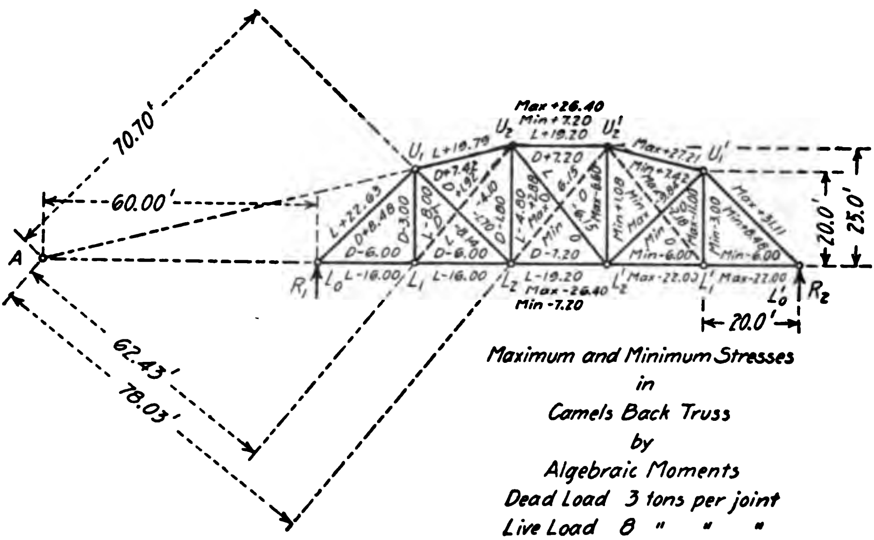



FIG. 7.

same in this truss as in a truss with parallel chords, and are as follows: Maximum chord stresses occur when all loads are on; minimum chord stresses occur when no live load is on; maximum web stresses in main members occur when the longer segment of the truss is loaded; and minimum stresses in main members and maximum stresses in counters occur when the shorter segment of the truss is loaded. For a proof of this criterion, see Fig. 6, Chapter IV. An apparent exception to the latter rule occurs in post $U_2L_2$, which has a maximum tensile stress when the truss is fully loaded with dead and live loads.

To calculate the stress in member $U_1L_2$, take moments about point $A$, the intersection of the upper and lower chords produced and pass a section cutting $U_1U_2$, $U_1L_2$ and $L_1L_2$, and cutting away the truss to the right. The dead load stress is then given by the equation

$$U_1L_2 \times 70.7 + R_1 \times 60 - W \times 80 = 0$$
$$U_1L_2 \times 70.7 = -6 \times 60 + 3 \times 80 = -120 \text{ foot-tons, and}$$
$$U_1L_2 = -1.70 \text{ tons.}$$

The maximum live load stress occurs when all loads are on except $L_1$, and

$$U_1L_2 \times 70.7 + R_1 \times 60 = 0$$
$$U_1L_2 \times 70.7 = -\tfrac{6}{8}P \times 60 = -576 \text{ foot-tons, and}$$
$$U_1L_2 = -8.14 \text{ tons}$$

The maximum live load stress in counter $U_2L_1$ occurs with a load at $L_1$, and is given by the equation

$$-U_2L_1 \times 62.43 + R_1 \times 60 - P \times 80 = 0$$
$$U_2L_1 \times 62.43 = \tfrac{7}{8}P \times 60 - 8 \times 80 = 256 \text{ foot-tons, and}$$
$$U_2L_1 = -4.10 \text{ tons}$$

The dead load stress in counter $U_2L_1$ when main member $U_1L_2$ is not acting will be

$$U_2L_1 \times 62.43 = +120 \text{ foot-tons, and}$$
$$U_2L_1 = +1.92 \text{ tons}$$

The maximum stress in $U_1L_2$ is therefore $-1.70 - 8.14 = -9.84$ tons, and the minimum stress is zero. The maximum stress in counter $U_2L_1$ is $+1.92 - 4.10 = -2.18$ tons, and the minimum stress is zero.

To calculate the stress in member $U_1U_2$, take the center of moments at $L_2$, and pass a section cutting $U_1U_2$, $U_2L_2$ and $L_2L_2'$, and cutting away the truss to the right. The dead load stress is then given by the equation

$$U_1U_2 \times 24.25 - R_1 \times 40 + W \times 20 = 0$$
$$U_1U_2 = +7.42 \text{ tons}$$

In like manner the live load stress in $U_1U_2 = +19.79$ tons.

The stresses in the remaining members may be found in the same manner. To obtain stress in upper chord $U_2U_2'$, take moments about $L_2$ as a center; to obtain stress in lower chord $L_0L_1$ take moments about $U_1$ as a center. The dead load and maximum live load tensile stress in post $U_2L_2$ is equal to the vertical component of the dead and live loads, respectively, in upper chord $U_1U_2$. The stresses in $L_0U_1$, $L_0L_1$, $L_2L_2'$, $U_2U_2'$ and $U_2L_2'$ are most easily found by algebraic resolution.

For additional problems, see Chapter VII.

**GRAPHIC MOMENTS.**—The dead load stresses in the chords of a Warren truss are calculated by graphic moments in Fig. 8.

**Bending Moment Polygon.**—The upper chord stresses are given by the ordinates to the bending moment parabola direct, while the lower chord stresses are arithmetical means of the upper chord stresses on each side, and are given by the ordinates to the chords of the parabola as shown in Fig. 8.

The parabola is constructed as follows: The mid-ordinate, 4–$j$, is made equal to the bending moment at the center of the truss divided by the depth; in this case the mid-ordinate is the stress in 6–$x$; if the number of panels in the truss were odd, the mid-ordinate would not be equal to any chord stress. The parabola is then constructed as shown in Fig. 8. The live load chord stresses may be found from Fig. 8 by changing the scale, or by multiplying the dead load chord stresses by a constant.

**Shear Polygon.**—In Chapter II it was shown that the maximum shear in a beam at any point could be represented by the ordinate to a parabola at the point. The same principle holds for a symmetrical bridge truss with equal panels and loaded with equal joint loads, as will now be proved.

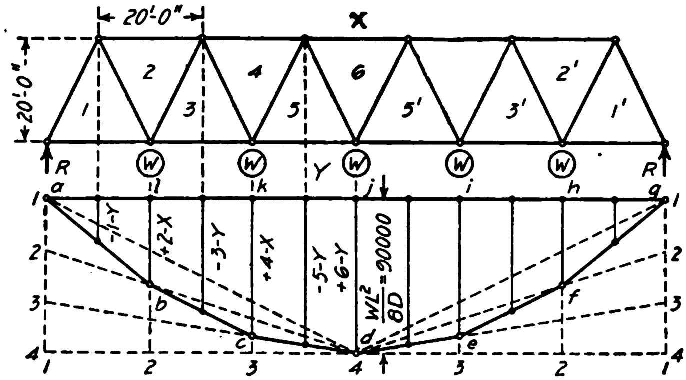


FIG. 8.

In Fig. 9 assume that the simple Warren truss is fixed at the left end as shown, and that the right reaction $R_2$ is not acting. Then with all joints fully loaded with a live load $P$, construct a

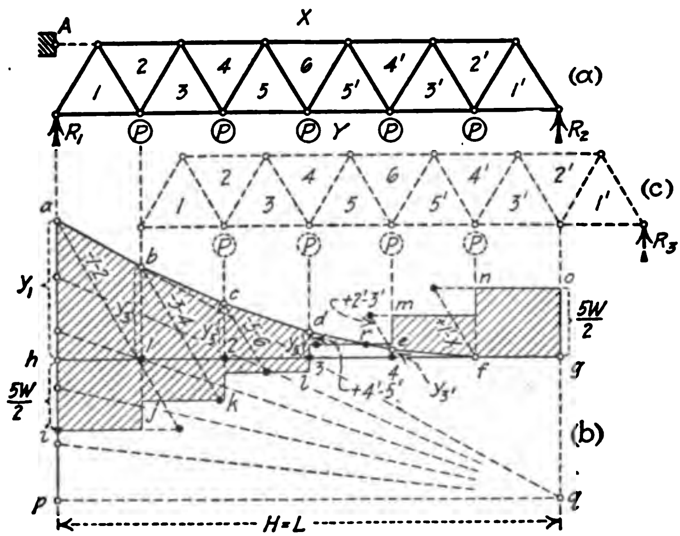


FIG. 9.

force polygon as shown, with pole $O$ and pole distance, $H$ = span $L$, and beginning at point $a$ in the load line of the force polygon, construct the equilibrium polygon $a$–$g$–$h$ for the cantilever truss.

Now the bending moment at the left support will be equal to ordinate $y_1$ multiplied by the pole distance $H$. But the truss is a simple truss and the moment of the right reaction will be equal to the moment at the left abutment, and

$$y_1 \cdot H = R_2 \cdot L$$

and since $H = L$

$$y_1 \cdot L = R_2 \cdot L$$

and

$$y_1 = R_2$$

Now, with the loads remaining stationary, move the truss one panel to the right as shown by the dotted truss. With the same force polygon draw a new equilibrium polygon as above. This equilibrium polygon will be identical with a part of the first equilibrium polygon as shown. As above, the bending moment at left reaction is $y_2 \cdot H = y_2 \cdot L = R_2 \cdot L$, and $y_2 = R_2$. In like manner $y_3$ can be shown to be the right reaction with three loads on, etc. Since the bridge is symmetrical with reference to the center line, the ordinates to the shear polygon in Fig. 9 are equal to the maximum shears in the panel to the right of the ordinate as the load moves off the bridge to the right.

For a method of drawing the shear parabola direct, without the use of the force and equilibrium polygons, see Problem 18, Chapter VII.

## CHAPTER IV.

### Stresses in Railway Bridge Trusses.

**LOADS.**—The dead load of a railway bridge is assumed to act at the joints the same as in a highway bridge. The dead joint loads are commonly assumed to act on the loaded chord, but may be assumed as divided between the panel points of the two chords, one-third and two-thirds of the dead loads usually being assumed as acting at the panel points of the unloaded and the loaded chords, respectively.

The live load on a railway bridge consists of wheel loads, the weights and spacing of the wheels depending upon the type of the rolling stock used. The locomotives and cars differ so much that it would be difficult if not impossible to design bridges on a railway system for the actual conditions, and conventional systems of loading, which approximate the actual conditions are assumed. The conventional systems for calculating the live load stresses in railway bridges that have been most favorably received are: (1) Cooper's Conventional System of Wheel Concentrations; (2) the use of an Equivalent Uniform Load; and (3) the use of a uniform load and one or two wheel concentrations. In addition to these some railroads specify special engine loadings. The first and second methods will be discussed in this chapter.

**Cooper's Conventional System of Wheel Concentrations.**—In Cooper's loadings two consolidation locomotives are followed by a uniformly distributed train load. The typical loading for Cooper's Class E 40 is shown in Fig. 3, Chapter IX. The loads on the drivers in thousands of pounds and the uniform train load in hundreds of pounds are the same as the class number. The wheel spacings are the same for all classes. The stresses for Cooper's loadings calculated for one class may be used to obtain the stresses due to any other class loading. For example, the stresses in any truss due to Cooper's Class E 50 are equal to $\frac{5}{4}$ of the stresses in the same truss due to Class E 40 loading. The E 55 and the E 60 loadings are those most used for steam railways in the United States. In bridges designed for Class E 40 loading and under the floor system must in addition be designed for two moving loads of 100,000 lb. each, spaced 6′ 0″ apart on each track. The corresponding loads for Class E 50 are 120,000 lb. with the same spacing. The American Railway Engineering Association has adopted Cooper's loadings, except that the special loads are spaced 7′ 0″. The values for moment, $M$, shear, $S$, and floorbeam reaction, $R$, for Class E 60 are given in Table I.

**Equivalent Uniform Load System.**—The equivalent uniform load for calculating the stresses in trusses and the bending moments in beams, is the uniform load that will produce the same bending moment at the quarter points of the truss or beam as the maximum bending moment produced by the wheel concentrations. The equivalent uniform loadings for different spans for Cooper's E 40 loading are given in Fig. 4, Chapter IX. In calculating the stresses in the truss members select the equivalent load for the given span, and calculate the chord and web stresses by the use of equal joint loads, as for highway bridges. In designing the stringers for bending moment take a loading for a span equal to one panel length, and for the maximum floorbeam reaction take a loading for a span equal to two panel lengths. It is necessary to calculate the maximum end shears and the shears at intermediate points by wheel concentrations, or to use equivalent uniform loads calculated for wheel concentrations.

Live load stresses calculated by the method of equivalent uniform loads are too small for the chords and webs between the ends of the truss and the quarter points, and are too large between the quarter points. The stresses obtained for the counters are too large. The live load

## TABLE I.

**Maximum Moments, M; End Shears, S; and Floorbeam Reactions, R; Per Rail, for Girders.**

Cooper's E60 Loading (A. R. E. A.).

Loading Two E 60 Engines and Train Load of 6,000 Pounds per Foot or Special Loading Two 75,000 Pound Axle Loads 7 Ft. C. to C.

Moments in Thousands of Foot-Pounds. Shears and Floorbeam Reactions in Thousands of Pounds.

Results for One Rail. Results from Special Loading marked*. A. R. E. A. Impact Formula.

| Span L. Ft. | Maximum Moments M. | Moment Impact M′. | End Shear S. | End Shear Impact S′. | Floorbeam Reaction R. | Floorbeam Impact R′. | Span L. Ft. | Maximum Moments M. | Moment Impact M′. | End Shear S. | End Shear Impact S′. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 5 | * 46.9 | * 46.1 | *37.5 | *36.9 | *37.5 | *36.3 | 50 | 1426.3 | 1222.6 | 130.8 | 112.1 |
| 6 | * 56.2 | * 55.1 | *37.5 | *36.8 | 40.0 | 38.5 | 51 | 1474.7 | 1260.4 | 132.5 | 113.2 |
| 7 | * 65.6 | * 64.2 | 38.6 | 37.7 | 47.1 | 45.0 | 52 | 1522.8 | 1297.8 | 134.1 | 114.3 |
| 8 | * 75.0 | * 73.0 | *42.2 | *41.2 | 52.5 | 49.8 | 53 | 1571.0 | 1335.1 | 135.7 | 115.3 |
| 9 | * 84.4 | * 82.0 | *45.8 | *44.5 | 56.7 | 53.5 | 54 | 1621.5 | 1374.2 | 137.4 | 116.4 |
| 10 | * 93.7 | * 90.7 | *48.8 | *47.2 | 60.0 | 56.3 | 55 | 1675.2 | 1415.7 | 139.0 | 117.5 |
| 11 | *103.0 | * 99.5 | *51.1 | *49.3 | 65.5 | 61.0 | 56 | 1728.0 | 1456.7 | 140.6 | 118.5 |
| 12 | 120.0 | 115.4 | *53.2 | *51.1 | 70.0 | 64.8 | 57 | 1781.9 | 1497.4 | 142.2 | 119.5 |
| 13 | 142.5 | 136.6 | 55.4 | 53.1 | 73.9 | 68.0 | 58 | 1834.5 | 1537.4 | 143.8 | 120.5 |
| 14 | 165.0 | 157.6 | 57.8 | 55.2 | 78.2 | 71.5 | 59 | 1891.4 | 1580.6 | 145.4 | 121.5 |
| 15 | 187.5 | 178.6 | 60.0 | 57.2 | 82.0 | 74.5 | 60 | 1949.4 | 1624.5 | 147.0 | 122.5 |
| 16 | 210.0 | 199.3 | 63.8 | 60.6 | 85.3 | 77.1 | 61 | 2007.5 | 1668.3 | 148.6 | 123.5 |
| 17 | 232.5 | 220.0 | 67.1 | 63.5 | 88.2 | 79.2 | 62 | 2064.3 | 1710.8 | 150.2 | 124.5 |
| 18 | 255.0 | 240.5 | 70.0 | 66.0 | 91.0 | 81.3 | 63 | 2123.4 | 1754.9 | 152.0 | 125.6 |
| 19 | 280.0 | 263.2 | 72.6 | 68.3 | 94.3 | 83.7 | 64 | 2183.3 | 1799.4 | 153.8 | 126.8 |
| 20 | 309.5 | 290.5 | 75.0 | 70.3 | 98.3 | 86.7 | 65 | 2246.3 | 1846.3 | 155.7 | 128.0 |
| 21 | 339.0 | 316.8 | 77.1 | 72.1 | 101.9 | 89.4 | 66 | 2309.3 | 1893.0 | 157.5 | 129.1 |
| 22 | 368.5 | 343.3 | 79.1 | 73.7 | 105.2 | 91.7 | 67 | 2378.3 | 1943.2 | 159.6 | 130.5 |
| 23 | 398.2 | 369.8 | 80.9 | 75.1 | 108.2 | 93.8 | 68 | 2435.4 | 1985.3 | 161.7 | 131.8 |
| 24 | 427.8 | 396.1 | 83.1 | 76.9 | 110.9 | 95.6 | 96 | 2498.4 | 2031.2 | 163.8 | 133.2 |
| 25 | 457.5 | 422.3 | 85.2 | 78.6 | 113.5 | 97.3 | 70 | 2561.3 | 2076.8 | 165.8 | 134.4 |
| 26 | 487.2 | 448.3 | 87.1 | 80.2 | 116.6 | 99.4 | 71 | 2624.5 | 2122.2 | 167.7 | 135.6 |
| 27 | 516.9 | 474.2 | 88.9 | 81.6 | 120.1 | 101.8 | 72 | 2688.0 | 2168.0 | 170.0 | 137.1 |
| 28 | 548.3 | 501.5 | 90.6 | 82.9 | 123.4 | 104.0 | 73 | 2750.9 | 2212.5 | 172.2 | 138.5 |
| 29 | 582.0 | 530.7 | 92.3 | 84.2 | 126.5 | 106.0 | 74 | 2818.5 | 2260.7 | 174.4 | 139.9 |
| 30 | 615.8 | 559.8 | 94.6 | 86.0 | 129.4 | 107.8 | 75 | 2888.6 | 2310.9 | 176.5 | 141.2 |
| 31 | 649.3 | 588.5 | 96.6 | 87.5 | 132.7 | 110.0 | 76 | 2958.0 | 2360.1 | 178.6 | 142.5 |
| 32 | 683.2 | 617.3 | 98.6 | 89.1 | 136.5 | 112.5 | 77 | 3028.6 | 2410.0 | 180.6 | 143.7 |
| 33 | 716.9 | 645.8 | 100.4 | 90.5 | 140.0 | 114.8 | 78 | 3096.6 | 2457.6 | 182.5 | 144.8 |
| 34 | 750.6 | 674.2 | 102.1 | 91.7 | 143.2 | 116.7 | 79 | 3168.2 | 2507.8 | 184.4 | 146.0 |
| 35 | 784.5 | 702.5 | 103.8 | 93.0 | 146.4 | 118.7 | 80 | 3240.7 | 2558.5 | 186.3 | 147.1 |
| 36 | 823.0 | 734.9 | 105.9 | 94.6 | 149.3 | 120.4 | 81 | 3311.4 | 2607.4 | 188.4 | 148.4 |
| 37 | 861.6 | 767.0 | 107.8 | 96.0 | 152.2 | 122.1 | 82 | 3385.1 | 2658.4 | 190.4 | 149.5 |
| 38 | 900.0 | 798.8 | 109.7 | 97.4 | 155.6 | 124.2 | 83 | 3459.6 | 2709.8 | 192.3 | 150.6 |
| 39 | 940.0 | 831.8 | 111.4 | 98.6 | 158.8 | 126.0 | 84 | 3534.6 | 2761.4 | 194.2 | 151.7 |
| 40 | 983.4 | 867.7 | 113.1 | 99.8 | 162.0 | 127.9 | 85 | 3610.4 | 2813.3 | 196.1 | 152.8 |
| 41 | 1027.0 | 903.5 | 115.2 | 101.3 | ......... | ......... | 86 | 3689.4 | 2867.4 | 198.1 | 154.0 |
| 42 | 1070.4 | 938.9 | 117.2 | 102.8 | ......... | ......... | 87 | 3766.5 | 2919.8 | 200.1 | 155.1 |
| 43 | 1113.9 | 974.2 | 119.0 | 104.1 | ......... | ......... | 88 | 3846.0 | 2973.7 | 202.1 | 156.3 |
| 44 | 1157.4 | 1009.4 | 120.8 | 105.3 | ......... | ......... | 89 | 3924.3 | 3026.5 | 204.0 | 157.3 |
| 45 | 1201.1 | 1044.4 | 122.5 | 106.5 | Viaduct | ......... | 90 | 4005.8 | 3081.4 | 205.8 | 158.3 |
| 46 | 1244.4 | 1078.9 | 124.2 | 107.7 | Span | ......... | 91 | 4084.4 | 3133.8 | 207.7 | 159.4 |
| 47 | 1287.9 | 1113.4 | 125.9 | 108.8 | 30′–60′ | ......... | 92 | 4164.0 | 3186.7 | 209.7 | 160.5 |
| 48 | 1331.4 | 1147.8 | 127.5 | 109.9 | 179.2 | ......... | 93 | 4246.6 | 3241.6 | 211.6 | 161.5 |
| 49 | 1378.3 | 1184.8 | 129.2 | 111.1 | ......... | ......... | 94 | 4328.0 | 3295.4 | 213.5 | 162.6 |

TABLE I.—*Continued.*

MAXIMUM MOMENTS, M; END SHEARS, S; AND FLOORBEAM REACTIONS, R; PER RAIL, FOR GIRDERS.

Cooper's E60 Loading (A. R. E. A.).

| Span L, Ft. | Maximum Moments M. | Moment Impact, M′. | End Shear S. | End Shear Impact S′. | Floorbeam Reaction R. | Floorbeam Impact R′. | Span L, Ft. | Maximum Moments M. | Moment Impact M′. | End Shear S. | End Shear Impact S′. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 95 | 4408.4 | 3348.2 | 215.4 | 163.6 | Viaduct | ......... | 110 | 5829.6 | 4265.5 | 243.0 | 177.8 |
| 96 | 4490.7 | 3402.0 | 217.2 | 164.5 | Span | ......... | 111 | 5937.4 | 4333.9 | 244.8 | 178.7 |
| 97 | 4573.5 | 3456.0 | 219.2 | 165.6 | 40′–60′ | ......... | 112 | 6040.0 | 4398.1 | 246.6 | 179.5 |
| 98 | 4659.8 | 3512.4 | 221.2 | 166.7 | 197.2 | ......... | 113 | 6148.2 | 4466.0 | 248.3 | 180.3 |
| 99 | 4743.8 | 3566.7 | 223.1 | 167.7 | ......... | ......... | 114 | 6258.0 | 4534.8 | 250.0 | 181.2 |
| 100 | 4830.0 | 3622.5 | 225.0 | 168.8 | Viaduct | ......... | 115 | 6366.8 | 4602.5 | 251.8 | 182.0 |
| 101 | 4916.9 | 3678.5 | 226.8 | 169.7 | Span | ......... | 116 | 6478.0 | 4671.6 | 253.6 | 182.9 |
| 102 | 5004.0 | 3734.4 | 228.6 | 170.6 | 40′–80′ | ......... | 117 | 6586.1 | 4738.2 | 255.3 | 183.6 |
| 103 | 5115.5 | 3808.1 | 230.4 | 171.5 | 236.5 | ......... | 118 | 6696.6 | 4806.1 | 257.0 | 184.4 |
| 104 | 5212.8 | 3870.9 | 232.3 | 172.5 | ......... | ......... | 119 | 6808.3 | 4874.7 | 258.8 | 185.3 |
| 105 | 5306.5 | 3930.7 | 234.1 | 173.4 | ......... | ......... | 120 | 6921.6 | 4944.0 | 260.5 | 186.1 |
| 106 | 5401.3 | 3991.1 | 235.9 | 174.3 | ......... | ......... | 121 | 7030.5 | 5009.9 | 262.2 | 186.9 |
| 107 | 5499.2 | 4053.4 | 237.7 | 175.2 | ......... | ......... | 122 | 7143.8 | 5078.5 | 264.0 | 187.7 |
| 108 | 5617.0 | 4130.1 | 239.4 | 176.0 | ......... | ......... | 123 | 7260.1 | 5148.9 | 265.7 | 188.4 |
| 109 | 5727.6 | 4201.1 | 241.2 | 176.9 | ......... | ......... | 124 | 7376.4 | 5219.1 | 267.4 | 189.2 |
| | | | | | | | 125 | 7495.2 | 5290.7 | 269.1 | 190.0 |

stresses calculated by the method of equivalent uniform loads are sufficiently accurate for all practical purposes. Even though the equivalent uniform load method is simple to apply and gives sufficiently accurate results, it is now seldom used.

**KINDS OF STRESS.**—The live loads on a railway bridge produce stresses as follows:

(1) Vertical stresses due to the live load in any position;

(2) Vibratory stresses due to the moving of the live load, generally included in the term "Impact";

(3) Horizontal static stresses due to centrifugal forces, if the train is on a curve;

(4) Longitudinal static stresses due to the momentum of the train, and the friction on the rails when the brakes are applied.

Vibratory stresses cannot be calculated with our present knowledge, but are provided for by taking a percentage of the static live load as "Impact Stress," or by using smaller working stresses. Horizontal and static stresses can be calculated.

**CALCULATION OF STRESSES DUE TO WHEEL CONCENTRATIONS.**—The maximum stresses in any member of a truss may be found by trial, that is, by assuming a number of positions of the live load, calculating the stress for each position, and then comparing the results. This method is long and tiresome and considerable time may be saved by the application of certain simple criteria, which will now be developed by means of influence diagrams. These criteria may also be developed by algebraic methods.

**INFLUENCE DIAGRAMS.**—An influence diagram (commonly called an influence line) shows the variation of the effect of a moving load or a system of loads on a beam or truss. The difference between bending moment or shear diagrams and influence diagrams is that the bending moment and the shear diagram gives the moment and shear, respectively, at any point for a fixed system of loads, while an influence diagram gives the moment or shear, etc., at a fixed point for a moving system of loads. Influence diagrams are used principally for finding the position of moving loads that will produce maximum shears, moments, reactions, or stresses, although they

may be used for calculating the quantities themselves. For convenience, where a number of loads are considered, the influence diagrams are drawn for a single unit load. The unit influence diagram may then be used for any load by multiplying by the given load. The unit influence diagram will be referred to in the following discussion.

**Maximum Moment in a Truss or Beam.**—Let $P_1$, in Fig. 1, represent the summation of the moving loads to the left of the panel point 2′, and $P_2$ be the summation of the moving loads to the right.

The influence diagram for the point 2′ is constructed by calculating the bending moment at 2′ due to a unit load $= a(L - a)/L$ = ordinate 2–4, and drawing lines 1–2 and 2–3. The equation of the line 1–2 is $y = x(L - a)/L$, and the equation of the line 2–3 is $y = a(L - x)/L$. Now when $x = a$ the two lines have a common ordinate which is equal to $a(L - a)/L$. Also when $x = L$ the ordinate to $1\text{–}2 = L - a$; while when $x = 0$, the ordinate to 2–3 is $a$, as is seen in Fig. 1. This relation gives an easy method of constructing an influence diagram for moments for any point in a beam or truss.

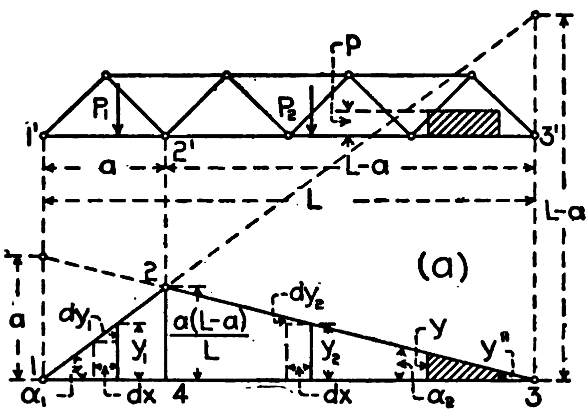



FIG. 1. INFLUENCE DIAGRAM FOR MOMENTS.

Now in Fig. 1 the bending moment at 2′ due to the loads $P_1$ and $P_2$, is

$$M = P_1 \cdot y_1 + P_2 \cdot y_2 \tag{1}$$

Now move the loads $P_1$ and $P_2$ a short distance to the left, the distance being assumed so small that the distribution of the loads will not be changed, and

$$M + dM = P_1(y_1 - dy_1) + P_2(y_2 + dy_2) \tag{2}$$

Subtracting (1) from (2) and placing $dM = 0$, we have

$$dM = - P_1 \cdot dy_1 + P_2 \cdot dy_2 = 0 \tag{3}$$

But $dy_1 = dx \cdot \tan \alpha_1 = dx(L - a)/L$, and $dy_2 = dx \cdot \tan \alpha_2 = dx \cdot a/L$, and

$$dM = - P_1(L - a)dx/L + P_2 \cdot a \cdot dx/L = 0, \text{ from which}$$

$$P_1 \cdot a - P_1 \cdot L + P_2 \cdot a = 0, \text{ and}$$

$$(P_1 + P_2)a = P_1 \cdot L$$

Solving, we have

$$P_1/a = (P_1 + P_2)/L. \tag{4}$$

*From (4) it follows that the maximum bending moment at 2′ occurs when the average load on the left of the section is the same as the average load on the entire bridge.* It will be seen that the criterion will be satisfied for a bridge loaded with equal joint loads when the bridge is fully loaded.

*Uniform Loads.*—In Fig. 1 the bending moment at 2′ due to a uniform load $p \cdot dx$ will be $p \cdot y \cdot dx$ in (a). But $y \cdot dx$ is the area of the influence diagram under the uniform load, and the bending moment at 2′ due to a uniform load will be equal to the area of the influence diagram covered by the load, multiplied by the load per unit of length. For a uniform load, $p$, covering the entire span the bending moment at 2′ will be $p$ times the area of the influence diagram 1–2–3. For a uniform load the bridge must be fully loaded to obtain maximum bending moment at any point. It will be seen that the general criterion for maximum bending moment is satisfied when the bridge is fully loaded with a uniform load.

**Maximum Shear in a Beam.**—It is required to calculate the maximum shear at the point 2′ in the beam 1′–4′, in Fig. 2. First consider two loads, $P_1$ and $P_2$, at a distance $b$ apart, on the

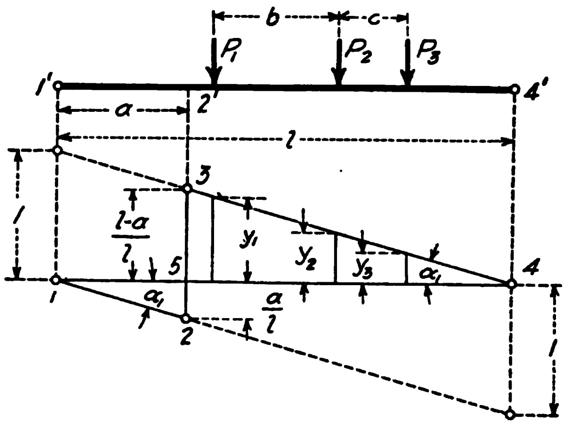


FIG. 2. INFLUENCE DIAGRAM FOR SHEAR IN A BEAM.

right of the point 2′. Now with the load unity at 2′ the shear at the left of the point will be $(l - a)/l$, and on the right of the point 2′ the shear will be $- a/l$, while the influence diagram for shear is 1–2–3–4. Then the shear at the point 2′ due to the loads $P_1$ and $P_2$ will be $S = + P_1 \cdot y_1 + P_2 \cdot y_2$. Now as the loads are moved to the left, the positive shear is increased until $P_1$ reaches point 2′. It will therefore be seen that a maximum shear occurs at 2′ when load $P_1$ is at 2′, and $P_2$ is on the longer segment of the beam. If $P_2$ is greater than $P_1$ the loads should be reversed in position.

For more than two loads, $P_1$, $P_2$ and $P_3$, the criterion is developed as follows:

The shear at 2′ is

$$S = + P_1 \cdot y_1 + P_2 \cdot y_2 + P_3 \cdot y_3 \tag{5}$$

Now move the loads to the left a distance $dx$, no loads coming on or going off the span, and

$$S + dS = + P_1(y_1 + dy_1) + P_2(y_2 + dy_2) + P_3(y_3 + dy_3) \tag{6}$$

subtracting (5) from (6), and solving for a maximum, we have

$$dS = + P_1 \cdot dy_1 + P_2 \cdot dy_2 + P_3 \cdot dy_3 = 0 \tag{7}$$

Now (7) can be satisfied only when $dy_1$, or both $dy_1$ and $dy_2$ are negative. Whether maximum shear will occur with $P_1$ or with $P_2$ at 2′ may be found by substituting in (5).

**Maximum Shear in a Truss.**—Let $P_1$, $P_2$ and $P_3$, in Fig. 3, represent the loads on the left of the panel, on the panel, and on the right of the $(m + 1)$st panel, respectively. It is required to find the position of the loads for a maximum shear in the panel.

With a load unity at 2′ the shear in the panel is $-m/n$, and 1–2 is the influence shear line for loads to the left of the panel. With a load unity at 3′ the shear in the panel is $(n - m - 1)/n$, and the line 3–4 is the influence shear line for loads to the right of the panel. For a load on the panel the shear will vary from $-m/n$ at 2′ to $(n - m - 1)/n$ at 3′, and the line 2–3 is the influence shear line for loads in the panel.

The influence diagram for the entire span is the polygon 1–2–3–4. It will be seen that the lines 1–2 and 3–4 are parallel, and are at a distance unity apart.

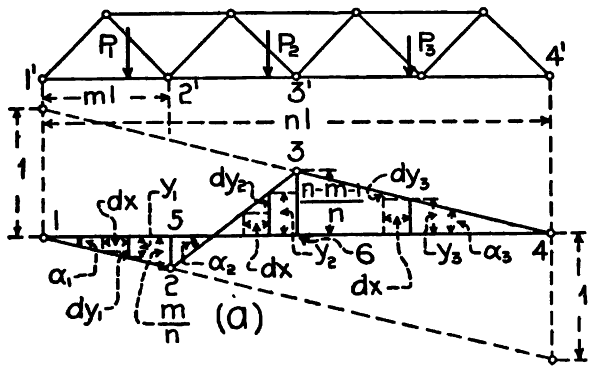



FIG. 3. INFLUENCE DIAGRAM FOR SHEAR IN A TRUSS.

The total shear in the panel will then be

$$S = -P_1 \cdot y_1 + P_2 \cdot y_2 + P_3 \cdot y_3 \tag{8}$$

Now move the loads a short distance to the left, the distance being assumed so small that the distribution of the loads will not be changed, and

$$S + dS = -P_1(y_1 - dy_1) + P_2(y_2 - dy_2) + P_3(y_3 + dy_3)$$

Subtracting (8) from (9), and solving for a maximum

$$dS = P_1 \cdot dy_1 - P_2 \cdot dy_2 + P_3 \cdot dy_3 = 0$$

But

$$dy_1 = dx \cdot \tan \alpha_1 = dx/n \cdot l,$$
$$dy_2 = dx \cdot \tan \alpha_2 = dx(n - 1)/n \cdot l,$$
$$dy_3 = dx \cdot \tan \alpha_3 = dx/n \cdot l;$$

and substituting we have

$$dS = P_1 \cdot dx/n \cdot l - P_2 \cdot dx(n - 1)/n \cdot l + P_3 \cdot dx/n \cdot l = 0$$
$$P_1 - P_2(n - 1) + P_3 = 0$$

and

$$P_1 + P_2 + P_3 = P_2 \cdot n$$

and

$$P_2 = (P_1 + P_2 + P_3)/n, \tag{10}$$

From (10) it follows that *the maximum shear in the panel will occur when the load on the panel is equal to the load on the bridge divided by the number of panels in the bridge.*

*Uniform Loads.*—In the same manner as for bending moment in Fig. 1, it can be proved that the shear in the panel due to a uniform load on the truss, in Fig. 3, is equal to the area of the influence diagram covered by the load, multiplied by the intensity of the uniform load per

linear unit. From Fig. 3 it will be seen that for a uniform load the maximum shear in the panel will occur when the uniform load extends from the right abutment to that point in the panel where the line 2–3 passes through the line 1–4 (where the shear changes sign). For a minimum shear in the panel (maximum shear of the opposite sign) the load should extend from the left abutment to the point in the panel where the shear changes sign. For equal joint loads, load the longer segment for a maximum shear in the panel, and load the shorter segment for a minimum shear in the panel.

**Maximum Floorbeam Reaction.**—It is required to find the maximum load on the floorbeam at 2′ in (a) Fig. 4 for the loads carried by the floor stringers in the panels 1′–2′ and 2′–3′.

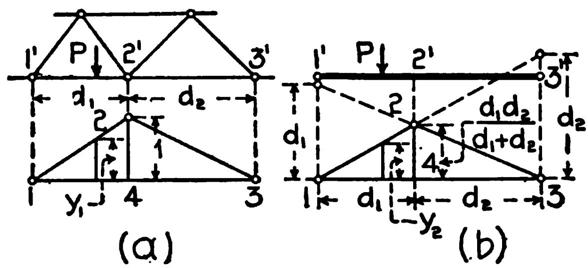



FIG. 4. INFLUENCE DIAGRAM FOR MAXIMUM FLOORBEAM REACTION.

In (a) the diagram 1–2–3 is the influence diagram for the shears at 2′ due to a unit load at any point in either panel. In (b) the diagram 1–2–3 is the influence diagram for bending moment at 2′ for a unit load at any point in the beam. Now the diagram in (a) differs from the diagram in (b), only in the value of the ordinate 2–4. It will be seen that the reaction at 2′ in (a) may be obtained from the diagram in (b), for any system of loads, if the ordinates are multiplied by $(d_1 + d_2)/d_1 \cdot d_2$. We can therefore use diagram (b) for obtaining the maximum floorbeam reaction, if we multiply all ordinates by $(d_1 + d_2)/d_1 \cdot d_2$.

*To obtain the maximum floorbeam reaction, therefore, take a simple beam equal to the sum of the two panel lengths, and find the maximum bending moment at a point in the beam corresponding to the panel point. This maximum moment multiplied by* $(d_1 + d_2)/d_1 \cdot d_2$ *will be the maximum floorbeam reaction.* If the two panels are equal in length the maximum bending moment at the center of the beam multiplied by $2/d$, where $d$ is the panel length, will give the maximum floorbeam reaction.

**Maximum Moment in the Unloaded Chord of a Through Warren Truss.**—Let $P_1$ in Fig. 5 represent the summation of the moving loads on the left of the panel 4′–5′, $P_2$ represent the summation of the moving loads on the panel, and $P_3$ represent the summation of the moving loads to the right of the panel. The influence diagram for the point 2 is the diagram 1–4–5–3, the lines 1–4 and 5–3 are the same as for a point on the loaded chord, while the influence line for the panel 4′–5′ is the line 4–5.

Now the bending moment at 2 due to the three loads is

$$M = P_1 \cdot y_1 + P_2 \cdot y_2 + P_3 \cdot y_3 \tag{11}$$

Now move the loads $P_1$, $P_2$, $P_3$ a short distance to the left, the distance being assumed so small that the distribution of the loads will not be changed, and

$$M + dM = P_1(y_1 - dy_1) + P_2(y_2 - dy_2) + P_3(y_3 + dy_3) \tag{12}$$

Subtracting (11) from (12), and solving for a maximum

$$dM = - P_1 \cdot dy_1 - P_2 \cdot dy_2 + P_3 \cdot dy_3 = 0 \tag{13}$$

Now $dy_1 = dx \cdot \tan \alpha_1$, $dy_2 = dx \cdot \tan \alpha_2$, and $dy_3 = dx \cdot \tan \alpha_3$.

$\tan\alpha_1 = (L - a)/L$, $\tan\alpha_3 = a/L$, and $\tan\alpha_2 = \dfrac{[L - (a - b + l)]\tan\alpha_3 - (a - b)\tan\alpha_1}{l}$
and $\tan\alpha_2 = (L\cdot b - a\cdot l)/L\cdot l$.

Substituting the values of $\tan\alpha_1$, $\tan\alpha_2$ and $\tan\alpha_3$ in (13) we have

$$- P_1(L - a)/L - P_2(L\cdot b - a\cdot l)/L\cdot l + P_3\cdot a/L = 0$$

Solving and placing $P = P_1 + P_2 + P_3$, we have

$$P/L = (P_1\cdot l + P_2\cdot b)/a\cdot l \tag{14}$$

Equation (14) is the criterion required.

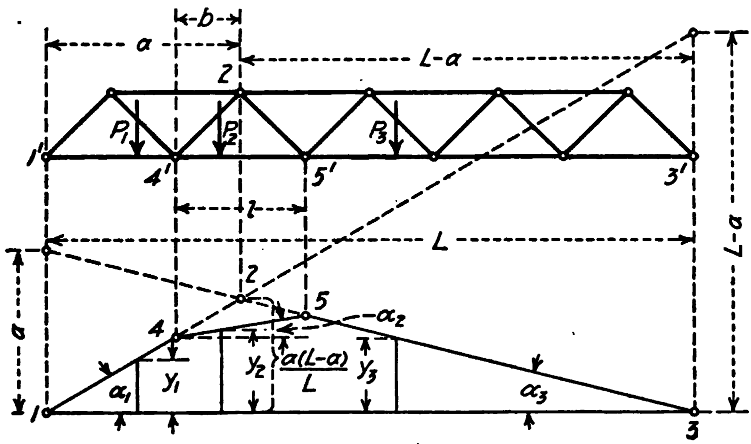



FIG. 5. INFLUENCE DIAGRAM FOR MOMENTS IN THE UNLOADED CHORD OF A THROUGH WARREN TRUSS.

**Maximum Stresses in a Bridge with Inclined Chords.**—Let $U_24'$ be a web member in a truss with inclined chords in Fig. 6. Point $A$ is the intersection of the upper chord $U_2U_3$ and the lower chord $2'4'$. The stress in $U_24'$ equals the moment of the external forces about the point $A$, divided by the arm $c$. The stress in the web member $U_24'$ will then be a maximum when the bending moment at $A$ is a maximum. To draw the moment influence diagram for the point $A$, calculate the bending moments about $A$ for the unit loads at $2'$ and $4'$. With a load unity at $4'$ the moment at $A$ is $(L - a - l)e/L$, and with a load unity at $2'$ the moment at $A$ is $(L - a)e/L - (a + e)$, a negative quantity. Laying off 4–6 and 2–7 equal to these moments, the influence diagram for bending moment at $A$ is the polygon 1–2–4–5.

The maximum stress in $U_24'$ occurs when some of the wheels at the head of the train are in the panel $2'4'$, and in unusual cases only, when a load is to the left of $2'$. Load $P_2$ representing the summation of the loads to the left of $4'$ will always come in the panel $2'4'$. Load $P_3$, representing the summation of the loads to the right of the panel, will always come to the right of the panel $2'4'$. Now the moment at $A$ is

$$M = P_2\cdot y_2 + P_3\cdot y_3 \tag{15}$$

Now move the loads a differential distance to the left, it being assumed that the distribution of the loads is not changed, and

$$M + dM = P_2(y_2 - dy_2) + P_3(y_3 + dy_3) \tag{16}$$

Subtracting (15) from (16), and solving for a maximum we have

$$dM = - P_2\cdot dy_2 + P_3\cdot dy_3 = 0 \tag{17}$$

Now $dy_2 = dx \cdot \tan \alpha_2$, and $dy_3 = dx \cdot \tan \alpha_3$, and

$$- P_2 \cdot \tan \alpha_2 + P_3 \cdot \tan \alpha_3 = 0 \tag{18}$$

and if $P = P_2 + P_3$,

$$P/(5-3) = P_2/(3-6) \tag{19}$$

Now $\tan \alpha_2 = -(l \cdot e/L - a - e)/l$, and $\tan \alpha_3 = e/L$, and substituting in (18) we have

$$P/L = P_2(1 + a/e)/l \tag{20}$$

which is the criterion required.

Now for a uniform load the maximum stress in the member $U_24'$ will occur when the truss is loaded from the right abutment to the point 3′, while the minimum stress will occur when the load extends from the left abutment to the point 3′. The critical point 3 can be calculated by drawing the lines $M$–2′–3′ and $N$–4′–3′. For wheel loads no load, should in general, pass 3′ from the right to give a maximum stress in the member.

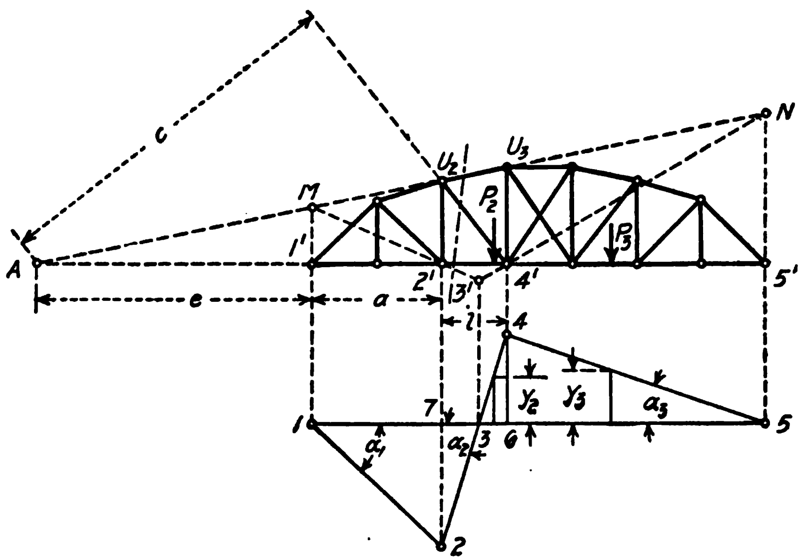


FIG. 6.

By substituting $e = \infty$ in (20) we have the criterion for maximum shear in a panel of a bridge with parallel chords.

**Resolution of the Shear.**—In Fig. 7 the stresses $U$, $D$ and $L$ hold in equilibrium the external forces on the left of the section cutting these members. These external forces consist of a left reaction, $R$, at the left abutment and a force at 2, equal to the reaction of the stringer 2–3. The resultant, $S$, of these two forces acts at a point a little to the left of the left reaction. Its position may be determined by moments. Referring to Fig. 7, let the resultant, $S$, be replaced by the two forces $P_1$ and $P_3$, $P_1$ acting upwards at 1 and $P_3$ acting downward at 3 as shown. Now taking moments about point 1, and

$$S \cdot a = P_3 \cdot l \tag{21}$$

Now the bending moment at 1 equals $M_1$, and

$$P_3 = S \cdot a/l = M_1/l \tag{22}$$

Similarly by taking moments at 3, we have

$$S(a + l) = P_1 \cdot l, \text{ but } S(a_1 + l) = M_3, \text{ and } P_1 = M_3/l$$

Now

$$S = P_1 - P_2 = M_1/l - M_3/l \tag{23}$$

where $S$ = shear in panel.

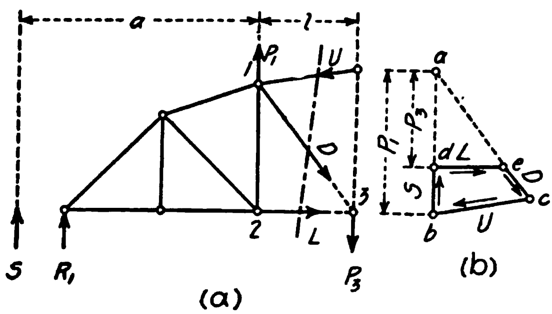



FIG. 7.

**Moment Diagram.**—A moment diagram for Cooper's Class E60 loading is given in Table II, the loading for maximum moment at joints in the loaded chord of truss bridges is given in Table III.

The details of the calculation of the stresses in a bridge with parallel chords is given in Problem 23, Chapter VII, and the details of the calculation of the stresses in a bridge with inclined chords is given in Problem 24, Chapter VII.

### POSITION OF WHEELS FOR MAXIMUM MOMENT; TABLE III.

The wheel loads that will produce maximum moment at a point a given distance from the left end of a beam, or at any loaded panel point in a bridge, are given in Table III. For example in an 8-panel Pratt truss of 200 ft. span, maximum moment at panel point $L_1$, 25 ft. from the left end, occurs with wheel No. 4, at the point; a maximum moment at $L_2$ occurs with wheel No. 7 at the point; etc.

### INSTRUCTIONS FOR USE OF MOMENT TABLE; TABLE II.

Line (1) is summation of loads from head of uniform load.
Line (2) is summation of loads from wheel No. 1.
Line (3) is the number of each wheel from wheel No. 1.
Line (4) is amount of each wheel load in thousand pounds.
Line (5) is distance c. to c. of the wheels, in feet.
Line (6) is distance of any wheel, or the head of uniform load, from wheel No. 1.
Line (7) is distance of any wheel from head of uniform load.
Line (8) is summation of moments of all wheels to right of any wheel, including the wheel in question, about head of uniform load.

Lines (9) to (25) are summations of moments of all wheels to left of the stepped line, including wheel on left of value, about the wheel just above the heavy vertical stepped line on each line.

The values to the right of the stepped lines are moments about the stepped line, including wheel to right of moment value given.

**EXAMPLES.—Problem 1.**—Calculate moment of wheels Nos. 1 to 15, inclusive, about wheel No. 15.

Follow vertical line passing through wheel No. 15 down to stepped line, and follow over to the left on line (12), and find 16,220 thousand ft.-lb. to right of vertical line through wheel No. 1.

**Problem 2.**—Calculate the moment of wheels Nos. 17, 16, 15, 14, about wheel No. 13.

Follow vertical line passing through wheel No. 13 down to the stepped line, and follow line (14) to right, and to left of the vertical line through wheel No. 17, find 1,281 thousand ft.-lb.

**Problem 3.**—Given a 200-ft. span, 8 panel Pratt railway bridge. The moments and shears are calculated as follows:

*Moments.*—Panel point $L_1$. From Table III, there will be a maximum moment at $L_1$ with wheel No. 4 at the joint; and from Table II, line (7) it is 91 ft. from wheel No. 4 to the end of the uniform load, and it is also 175 ft. from joint $L_1$ to the end of the bridge, and there will be $175 - 91 = 84$ ft. of uniform load on the bridge. Then, $R_1 \times 200 = 24{,}550 + 426 \times 84 + 3 \times 84^2/2 = 70{,}918$ thousand ft.-lb.; and $R_1 = 354.6$ thousand lb. The moment at $L_1$ is $M_1 = 354.6 \times 25 - 720 = 8{,}145$ thousand ft.-lb.

*Shear in Panel* $L_0L_1$ is $S_1 = R_1 - 720/25 = 354.6 - 28.8 = 325.8$ thousand lb. (720 is the moment of wheels Nos. 1, 2, 3, about wheel No. 4).

## TABLE III.

### LOADING FOR MAXIMUM MOMENT IN BRIDGES FOR COOPER'S LOADINGS.

| Spans | 10' | 15' | 20' | 25' | 30' | 35' | 40' | 45' | 50' | 55' | 60' | 65' | 70' | 80' | 90' | 100' | 110' | 120' | 130' | 140' |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 300' to 260' | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 18 |
| 250' to 200' | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 17 | 18 |
| 190' to 150' | 2 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 9 | 11 | 12 | 12 | 13 | 14 | 15 | 17 | 18 |
| 140' | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 12 | 13 | 14 | 15 | 17 | 18 |
| 130' | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 12 | 13 | 14 | 15 | 17 | |
| 120' | 3 | 3 | 3 | 4 | 4 | 5 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 13 | 14 | 15 | | |
| 110' | 3 | 3 | 3 | 4 | 4 | 5 | 6 | 7 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 13 | 14 | | | |
| 100' | 3 | 3 | 3 | 4 | 5 | 5 | 6 | $\overline{14}$ | $\overline{14}$ | $\overline{14}$ | $\overline{13}$ | $\overline{13}$ | 11 | 12 | 13 | 13 | | | | |
| 90' | 3 | 3 | 4 | 4 | 5 | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | 12 | 13 | | | | | |
| 80' | 3 | 3 | 4 | 4 | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | 12 | | | | | | |
| 70' | 3 | 3 | 4 | 4 | $\overline{13}$ | $\overline{13}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{11}$ | $\overline{11}$ | 11 | | | | | | | |
| 65' | 3 | 3 | 4 | 4 | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{12}$ | $\overline{11}$ | $\overline{11}$ | 11 | | | | | | | | |
| 60' | 3 | 3 | 4 | 4 | 5 | $\overline{13}$ | $\overline{12}$ | $\overline{11}$ | $\overline{11}$ | $\overline{11}$ | 11 | | | | | | | | | |
| 55' | 12 | 12 | 12 | 4 | 12 | $\overline{13}$ | $\overline{12}$ | $\overline{12}$ | $\overline{13}$ | 11 | | | | | | | | | | |
| 50' | 12 | 12 | 12 | 12 | 12 | $\overline{13}$ | $\overline{13}$ | $\overline{13}$ | 12 | | | | | | | | | | | |
| 45' | 12 | 12 | 12 | 12 | 12 | $\overline{13}$ | $\overline{13}$ | 13 | | | | | | | | | | | | |
| 40' | 3 | 3 | 3 | 12 | 12 | $\overline{13}$ | 13 | | | | | | | | | | | | | |
| 35' | 3 | 3 | 4 | 4 | 13 | 13 | | | | | | | | | | | | | | |
| 30' | 3 | 3 | 4 | 4 | 13 | | | | | | | | | | | | | | | |
| 25' | 3 | 3 | 4 | 4 | | | | | | | | | | | | | | | | |
| 20' | $\overline{4}$ | 3 | 4 | | | | | | | | | | | | | | | | | |
| 15' | 3 | 3 | | | | | | | | | | | | | | | | | | |
| 10' | 3 | | | | | | | | | | | | | | | | | | | |

WHEEL DETERMINING MAXIMUM MOMENT
COOPER'S LOADINGS
C.M.& ST.P.RY.

The shorter span is ahead followed by the longer one except wheel is over-lined.

TABLE II.

MOMENT TABLE FOR COOPER'S E60 LOADING.

| | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 426 | 411 | 381 | 351 | 321 | 291 | 271.5 | 252 | 232.5 | 213 | 198 | 168 | 138 | 108 | 78 | 58.5 | 39 | 19.5 | |
| 2 | 15 | 45 | 75 | 105 | 135 | 154.5 | 174 | 193.5 | 213 | 228 | 258 | 288 | 318 | 348 | 367.5 | 387 | 406.5 | 426 | |
| 3 | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 14 | 15 | 16 | 17 | 18 | Uniform Load 3000# per lin. Ft. |
| 4 | 15 | 30 | 30 | 30 | 30 | 19.5 | 19.5 | 19.5 | 19.5 | 15 | 30 | 30 | 30 | 30 | 19.5 | 19.5 | 19.5 | 19.5 | |
| 5 | 8' | 5' | 5' | 5' | 9' | 5' | 6' | 5' | 8' | 8' | 5' | 5' | 5' | 9' | 5' | 6' | 5' | 5' | |
| 6 | 8' | 13' | 18' | 23' | 32' | 37' | 43' | 48' | 56' | 64' | 69' | 74' | 79' | 88' | 93' | 99' | 104' | 109' | |
| 7 | 109' | 101' | 96' | 91' | 86' | 77' | 72' | 66' | 61' | 53' | 45' | 40' | 35' | 30' | 21' | 16' | 10' | 5' | |
| 8 | 24550 | 22910 | 19880 | 17000 | 14270 | 11690 | 10190 | 8790 | 7500 | 6310 | 5514 | 4164 | 2965 | 1914 | 1014 | 605 | 292.5 | 97.5 | |
| 9 | 22420 | 20860 | 17980 | 15250 | 12670 | 10240 | 8830 | 7530 | 6340 | 5240 | 4524 | 3325 | 2275 | 1374 | 624 | 312 | 97.5 | | |
| 10 | 20380 | 18900 | 16170 | 13590 | 11160 | 8880 | 7570 | 6360 | 5270 | 4280 | 3632 | 2580 | 1682 | 932 | 331.5 | 117 | 97.5 | | |
| 11 | 18060 | 16670 | 14120 | 11720 | 9470 | 7370 | 6180 | 5090 | 4110 | 3230 | 2678 | 1808 | 1088 | 518 | 97.5 | 117 | 331.5 | | |
| 12 | 16220 | 14900 | 12500 | 10250 | 8155 | 6200 | 5110 | 4120 | 3240 | 2460 | 1980 | 1260 | 690 | 270 | 97.5 | 312 | 624 | | |
| 13 | 13090 | 11910 | 9780 | 7800 | 5970 | 4290 | 3370 | 2550 | 1850 | 1245 | 900 | 450 | 150 | 175.5 | 448.5 | 838.5 | 1326 | | |
| 14 | 11500 | 10400 | 8410 | 6580 | 4900 | 3370 | 2555 | 1830 | 1227 | 720 | 450 | 150 | 150 | 423 | 793.5 | 1281 | 1866 | | |
| 15 | 10060 | 9030 | 7200 | 5520 | 3990 | 2605 | 1885 | 1262 | 755 | 345 | 150 | 150 | 450 | 820.5 | 1288.5 | 1873 | 2556 | | |
| 16 | 8770 | 7810 | 6130 | 4600 | 3220 | 1992 | 1368 | 842 | 432 | 120 | 150 | 450 | 900 | 1368 | 1933.5 | 2620 | 3396 | | |
| 17 | 6950 | 6110 | 4670 | 3380 | 2240 | 1248 | 780 | 409.5 | 156 | 240 | 630 | 1170 | 1860 | 2484 | 3705.5 | 4044 | 4980 | | |
| 18 | 5240 | 4524 | 3325 | 2275 | 1374 | 624 | 312 | 97.5 | | | | | | | | | | | |
| 19 | 4280 | 3632 | 2580 | 1682 | 932 | 331.5 | 117 | | | | | | | | | | | | |
| 20 | 3230 | 2678 | 1808 | 1088 | 518 | 97.5 | | | | | | | | | | | | | |
| 21 | 2460 | 1980 | 1260 | 690 | 270 | | | | | | | | | | | | | | |
| 22 | 1245 | 900 | 450 | 150 | | | | | | | | | | | | | | | |
| 23 | 720 | 450 | 150 | | | | | | | | | | | | | | | | |
| 24 | 345 | 150 | | | | | | | | | | | | | | | | | |
| 25 | 120 | | | | | | | | | | | | | | | | | | |

MOMENT TABLE
COOPER'S E-60 LOADING
TWO 213 TON ENGINES + 6000 LBS. PER FOOT.
MOMENT IN THOUSAND FOOT POUNDS FOR ONE RAIL.
LOADS IN THOUSANDS OF POUNDS FOR ONE RAIL.

# CHAPTER V.

## Stresses in Lateral Systems.

**Introduction.**—The wind loads on bridges are carried to the abutments by the lateral systems. In a through truss bridge the lateral systems usually consist of the top lateral system, the bottom lateral system, the intermediate bents or sway bracing between the intermediate posts, and the portals in the planes of the end-posts as shown in Fig. 1, Chapter VIII. In shallow through truss bridges the sway bracing is sometimes omitted; in deck trusses the portals are replaced by sway bracing; while in low trusses the bottom lateral system only is used.

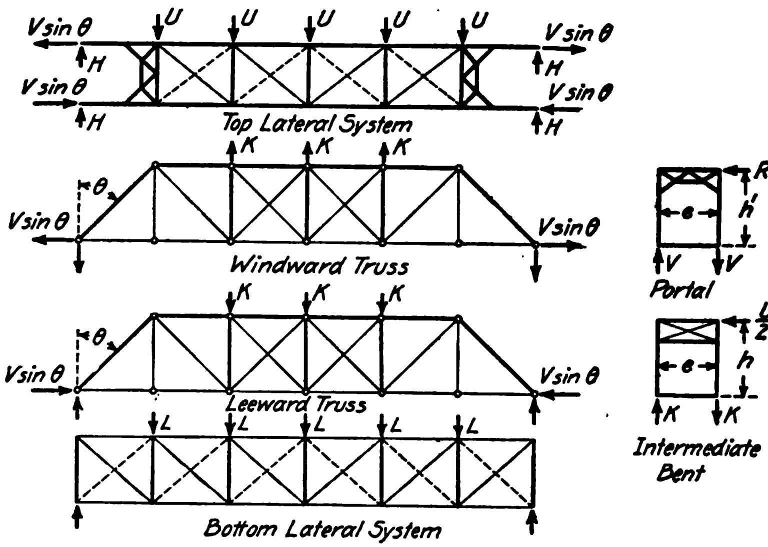



Fig. 1.

**Wind Loads.**—The wind loads are usually given in specifications as a certain number of pounds per lineal foot of bridge or per square foot of exposed surface. The wind load is usually taken at 30 lb. per square foot of exposed surface when the live load is on the bridge and at 50 lb. per square foot of exposed surface when the bridge is unloaded.

The usual specification for highway bridges is: A wind load of 150 lb. per lineal foot of bridge on the unloaded chord to be treated as a dead load, and a wind load of 300 lb. per lineal foot of bridge on the loaded chord, 150 lb. of which is to be treated as a dead load and 150 lb. to be treated as a live load. In railroad bridges the dead load wind is usually taken the same as for highway bridges, while the live load wind is taken at 450 lb. to 600 lb. per lineal foot. For extracts from standard specifications, see Chapter IX.

**STRESSES IN LATERAL SYSTEMS.**—In the through Pratt truss bridge in Fig. 1 the wind joint loads on the upper chord are equal to $U$, while the joint loads on the lower chord are equal to $L$. Where sway bracing is used, part of the upper chord loads are transferred to the lower

lateral system by the sway bracing. The exact amount thus transferred is statically indeterminate, but is usually assumed as $U/2$ at all joints having sway bracing. This load $U/2$ produces a vertical load, $K$, at each joint in the vertical trusses, which acts downward on the leeward and upward on the windward side of the bridge. Each portal transfers the load carried to the hip

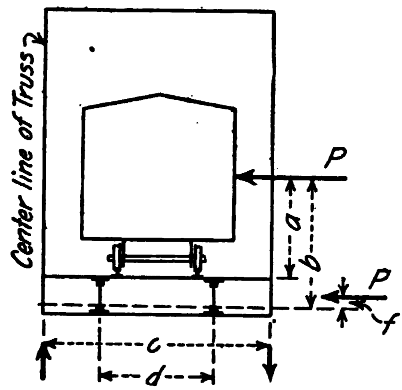



FIG. 2.

joint by the upper lateral system, and the load at the hip joint to the abutments. This produces a tension $V \cdot \sin\theta$ in the bottom chord on the leeward side and a compression $V \cdot \sin\theta$ in the bottom chord on the windward side.

In addition to the wind loads on the top chord that are transferred to the bottom lateral system, the wind load on the train of cars on steam and electric railway bridges, increases the loading on the vertical trusses on the leeward side and decreases the loading on the windward side of the bridge as shown in Fig. 2. This increase or decrease in vertical loading can be calculated by taking moments about the line of the pins in the lower chord. The wind load acting on the train is usually specified as applied six feet above the base of the rail.

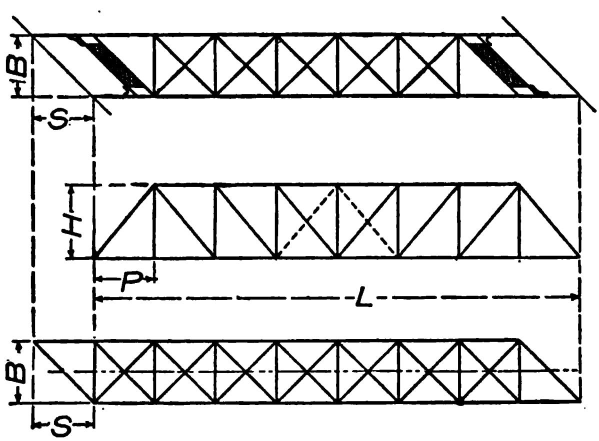



FIG. 3. SKEW BRIDGE.

**Skew Bridge.**—In a skew bridge the abutments are not at right angles to the center line of the bridge. This gives a warped portal, and ties which are not vertical unless the bridge is skewed one entire panel as is shown in Fig. 3, which is the common practice.

**Initial Stresses.**—In (a) Fig. 4 the diagonal lateral rods have an initial stress of 10,000 lb. in each rod. In (b) the lateral truss is loaded with loads of 12,000 lb. at joints $B$, $C$ and $D$, producing stresses as shown. In (c) the combined stresses due to direct loads and the initial stresses are shown. The stresses in the chords and struts can now be calculated by algebraic resolution. The stresses are combined as follows: In panel $B$–$C$ each rod has an initial stress of 10,000 lb., and in addition must transfer a wind shear of 6,000 lb. or an inclined stress of 9,000 lb. Half of the 9,000 lb. or 4,500 lb. will be added to the initial stress in $Bc$, making the stress $-$ 14,500 lb.,

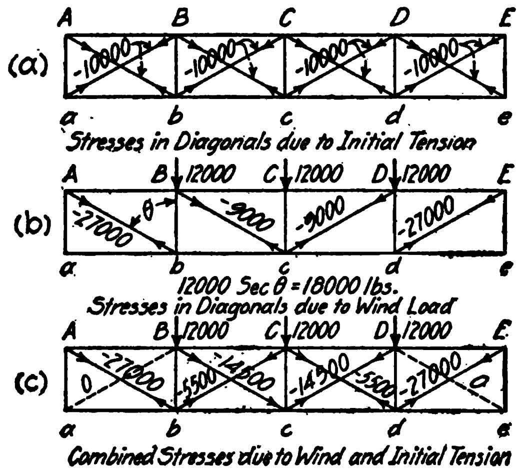



FIG. 4.

while 4,500 lb. will be subtracted from the, initial stress in $Cb$, making the stress $-$ 5,500 lb. In panel $A$–$B$ the initial stress in $aB$ is entirely relieved by the direct stress, while the initial stress in $Ab$ is increased by the direct stress remaining after the initial stress of 10,000 lb. was relieved in $aB$, or 17,000 lb., making the total stress in $Ab = -10{,}000 - 17{,}000 = -27{,}000$ lb., the same as if there had been no initial stress in the panel. This solution is based on the mathematical principle "*That if a load may be carried from one point to another by more than one route, it will be divided between the routes in proportion to the rigidities of the routes.*" In the problem above the two routes are assumed to have the same rigidities.

**PORTALS.**—Portal bracing is placed at the ends of through bridges in the planes of the end-posts to transfer the wind loads from the upper lateral system to the abutments. The stresses in the sway bracing placed in the planes of the intermediate posts are calculated in the same manner as the stresses in portal bracing. Portal bracing should be designed so that the stresses will be statically determinate. Several different types of portals are shown in Fig. 5. Types (a), (b) and (d) are the types most used for highway bridges. The lower ends of the end-posts may be hinged (free to turn), or fixed. The criterion for determining whether the end-posts are fixed or not will be discussed in Chapter VI.

**Case I. Stresses in Simple Portals:** *End-posts Hinged.*—The deflections of the posts in the portals shown in Fig. 5 are assumed to be equal, and

$$H = H' = R/2$$

Taking moments about the foot of the windward post

$$V' = -V = R \cdot h/s \tag{1}$$

Having found the external forces, the stresses in the members may be found by either algebraic or graphic methods.

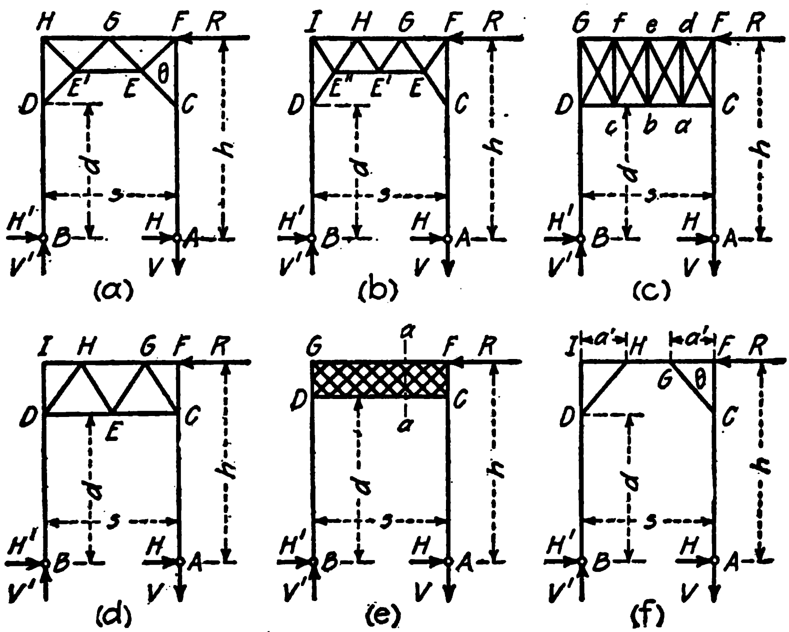



FIG. 5. TYPES OF PORTALS.

**Algebraic Solution.** *Portal* (*a*).—To obtain the stress in member $G$–$C$, (a) Fig. 5, pass a section cutting $G$–$F$, $E$–$F$ and $G$–$C$, and take moments of the external forces to the right of the section, about point $F$ as a center.

$$G\text{–}C = -H\cdot h/[(h-d)\sin\theta] \tag{2}$$

But $H = R/2$, and $(h-d)\sin\theta = \frac{1}{2}s\cdot\cos\theta$. Substituting these values in (2) we have

$$G\text{–}C = -R\cdot h/(s\cdot\cos\theta) = -V\cdot\sec\theta \tag{3}$$

Resolving at $C$ and $F$ we have, stress in $E$–$F$ = 0, and also stresses $E$–$E'$ and $H$–$E'$ = 0.

To obtain stress in $G$–$D$, pass section cutting $H$–$G$, $H$–$E'$ and $G$–$D$, and take moments of the external forces to the left of the section, about point $H$ as a center.

$$G\text{–}D = H\cdot h/[(h-d)\sin\theta] = +V\cdot\sec\theta \tag{4}$$

To obtain stress in $G$–$F$, pass a section cutting $G$–$F$, $E$–$F$ and $G$–$C$, and take moments of the external forces to the right of the section, about point $C$ as a center

$$G\text{–}F = +[R(h-d) + H\cdot d]/(h-d) \tag{5}$$

To obtain stress in $H$–$G$, pass a section cutting $H$–$G$, $H$–$E'$ and $G$–$D$, and take moments of the external forces to the left of the section, about the point $D$ as a center.

$$H\text{–}G = -H\cdot d/(h-d) \tag{6}$$

The stress in the windward post, $A$–$F$, is zero above and $V$ below the foot of the knee brace $C$; the stress in the leeward post is zero above and $V'$ below the foot of the knee brace $D$.

The shear in the posts is $H$ below the foot of the knee brace, and above the foot of the knee brace is given by the formula

$$S = H\cdot d/(h-d) = \text{stress in } H\text{–}G \tag{7}$$

The maximum moment in the posts occurs at the foot of the knee braces $C$ and $D$, and is

$$M = H \cdot d \tag{8}$$

For the actual stresses, moments and shears in a portal of this type, see Fig. 6.

*Portal* (*b*).—The stresses in portal (b) Fig. 5, are found in the same manner as in portal (a). The graphic solution of a similar portal with one more panel is given in Fig. 7, which see. It should be noted that all members are stressed in portals (b) and (d).

*Portal* (*c*).—The stresses in portal (c) Fig. 5, may be obtained (1) by separating the portal into two separate portals with simple bracing, the stresses found by calculating the separate simple portals with a load $= \frac{1}{2}R$, being combined algebraically, to give the stresses in the portal; or (2) by assuming that the stresses are all taken by the system of bracing in which the diagonal ties are in tension. The latter method is the one usually employed and is the simpler.

Maximum moment, shear and stresses in the posts are given by the same formulas as in (a) Fig. 5.

*Portal* (*e*).—In portal (e) Fig. 5, the flanges $G–F$ and $D–C$ are assumed to take all the bending moment, and the lattice web bracing is assumed to take all the shear. The maximum compression in the upper flange $G–F$ occurs at $F$, and is

$$G–F = + [R(h - d) + H \cdot d]/(h - d) \tag{9}$$

The maximum tension in the upper flange $G–F$ is

$$G–F = - H \cdot d/(h - d) \tag{10}$$

The maximum stress in the lower flange $D–C$ is

$$D–C = \pm H \cdot h/(h - d) \tag{11}$$

maximum tension occurring at $C$, and maximum compression occurring at $D$.

The maximum shear in the portal strut is $V$, which is assumed as taken equally by the lattice members cut by a section, as $a–a$.

Maximum moment, shear and stresses in the posts are given by the same formulas as in (a) Fig. 5.

*Portal* (*f*).—The maximum moment in the portal strut $I–F$ in (f) Fig. 5, occurs at $H$ and $G$, and is

$$M = + H \cdot h - V \cdot a \tag{12}$$

The maximum direct stress in $H–G$ is $+ H$, and in $I–H$ is

$$I–H = - H \cdot d/(h - d) \tag{13}$$

The maximum stress in $G–F$ is given by formula (5).

The maximum shear in girder $I–F$ is equal to $V$. The stress in $G–C$ is $- V \cdot \sec \theta$ and in $H–D$ is $+ V \cdot \sec \theta$, as in (a) Fig. 5.

Portal strut $I–F$ is designed as a girder to take the maximum moment, shear and direct stress.

Maximum moment, shear and stresses in the posts are given by the same formulas as in (a) Fig. 5.

**Graphic Solution.**—To make the solution of the stresses statically determinate, replace the posts in the portals with trussed framework as in Fig. 6. The stresses in the interior members are not affected by substituting the dotted members, and will be correctly given by graphic resolution.

As before $H = H' = R/2$ and $V = - V' = R \cdot h/s$.

Having the calculated $H$, $H'$, $V$ and $V'$, the stresses are calculated by graphic resolution as follows: Beginning at the base of the column $A$, lay off $A–4 = V = 3{,}000$ lb. acting downward, and $A–a = H = 1{,}000$ lb. acting to the right. Then $a–1$ and $4–1$ are the stresses in members $a–1$ and $4–1$, respectively, heavy lines indicating compression and light lines tension. At joint in

auxiliary truss to right of $C$ the stress in 1–$a$ is known and stresses in 1–2 and 2–$a$ are found by closing the polygon. The stresses in the remaining members are found in like manner, taking joints $C$, $E$, $F$, etc., in order, and finally checking up at the base of the post $B$. The full lines in the stress diagram represent stresses in the portal; the dotted lines represent stresses in the auxiliary members or stresses in members due to auxiliary members, and are of no consequence. The shears and moments are shown in the diagram.

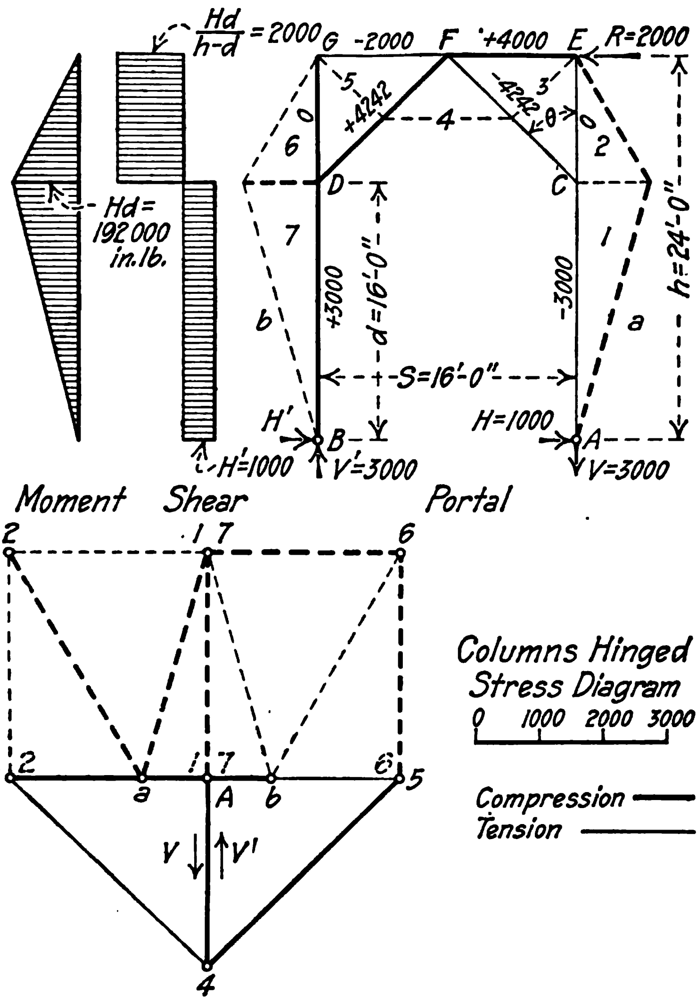



FIG. 6.

**Simple Portal as a Three-Hinged Arch.**—In a simple portal the resultant reactions and the external load, $R$, meet in a point at the middle of the top strut, and the portal then becomes a three-hinged arch ("Design of Steel Mill Buildings," Chapter XIII), provided there is a joint at that point (point $b$, Fig. 7).

In Fig. 7 the reactions were calculated graphically and the stresses in the portal were calculated by graphic resolution. Full lines in the stress diagram represent required stresses in the

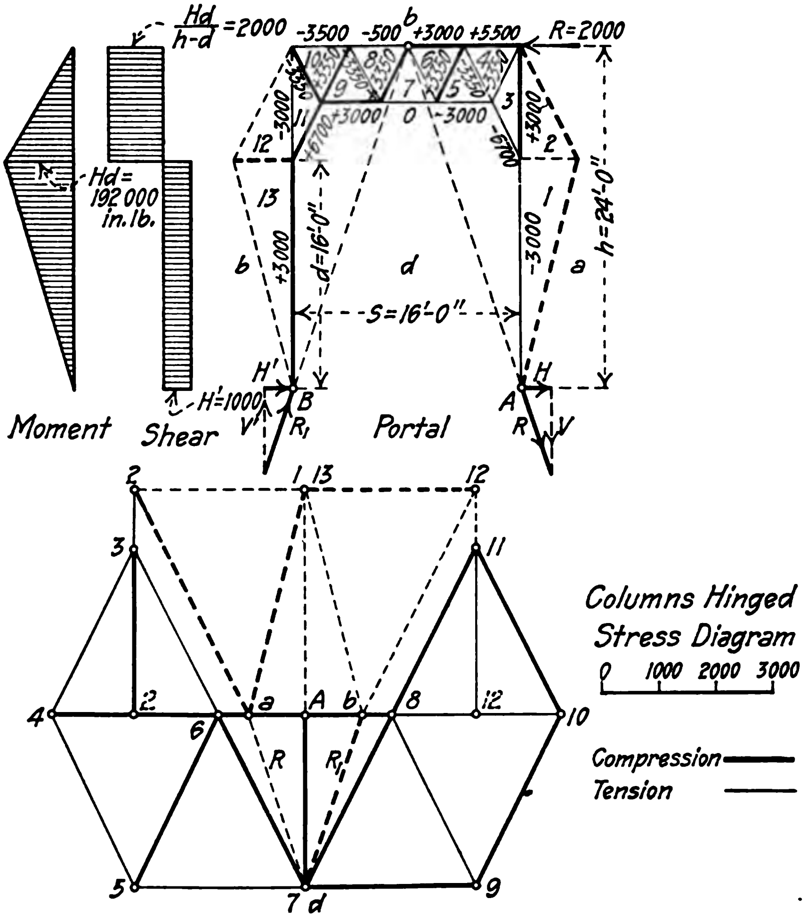



FIG. 7.

members. Stresses 3–2 and 11–12 were determined by dropping verticals from points 3 and 11 to the load line 4–10.

**Case II. Stresses in Simple Portals.** *Posts Fixed.*—The calculation of the stresses in a portal with posts fixed at the base is similar to the calculation of stresses in a transverse bent with columns fixed at the base.* The point of contra-flexure is at the point

$$y_0 = (d/2)\frac{d + 2h}{2d + h} \tag{14}$$

measured up from the base of the post. The point of contra-flexure is usually taken at a point a distance $d/2$ above the bases of the posts.

* See the author's book "The Design of Steel Mill Buildings," Chapter XI.

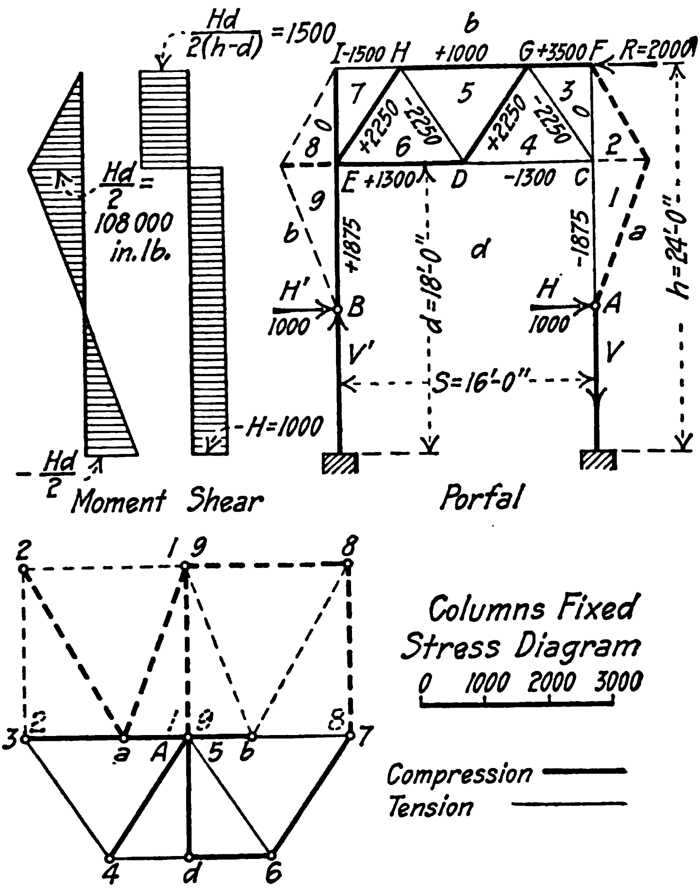



FIG. 8.

The stresses in a portal with posts fixed may be calculated by considering the posts hinged at the point of contra-flexure and solving as in Case 1.

**Algebraic Solution.**—In Fig. 8 we have

$$H = H' = R/2$$

and

$$V = -V' = \frac{R(2h-d)}{2s} \tag{15}$$

Having found the reactions $H$ and $H'$, $V$ and $V'$, the stresses in the members are found by taking moments as in (a) Fig. 5, considering the posts as hinged at the point of contra-flexure. The shear diagram for the posts is as shown in (a) and the moment diagram as in (c), Fig. 8.

**Graphic Solution.**—The stresses in the portal in Fig. 8 have been calculated by graphic resolution. This problem is solved in the same manner as the simple portal with hinged posts in Fig. 6.

For the calculation of the stresses in a portal, see Problem 25, Chapter VII.

# CHAPTER VI.

## Stresses in Pins, Eccentric and Combined Stresses, Deflection of Trusses, Stresses in Rollers, and Camber.

**STRESSES IN PINS.**—A pin under ordinary conditions is a short beam and must be designed (1) for bending, (2) for shear, and (3) for bearing. If a pin becomes bent the distribution of the loads and the calculation of the stresses are very uncertain.

The cross-bending stress, $S$, is found by means of the fundamental formula for flexure, $S = M \cdot c/I$, where the maximum bending moment, $M$, is found as explained later; $I$ is the moment of inertia; and $c$ is one-half the diameter of a solid or hollow pin.

The safe shearing stresses given in standard specifications are for a uniform distribution of the shear over the entire cross-section, and the actual unit shearing stress to be used in designing will be equal to the maximum shear divided by the area of the cross-section of the pin.

The bearing stress is found by dividing the stress in the member by the bearing area of the pin, found by multiplying the thickness of the bearing plates by the diameter of the pin.

**Calculation of Stresses.**—The method of calculation will be illustrated by calculating the stresses in the pin at $U_1$ in (a) Fig. 1. In the complete investigation of the pin $U_1$, it would be

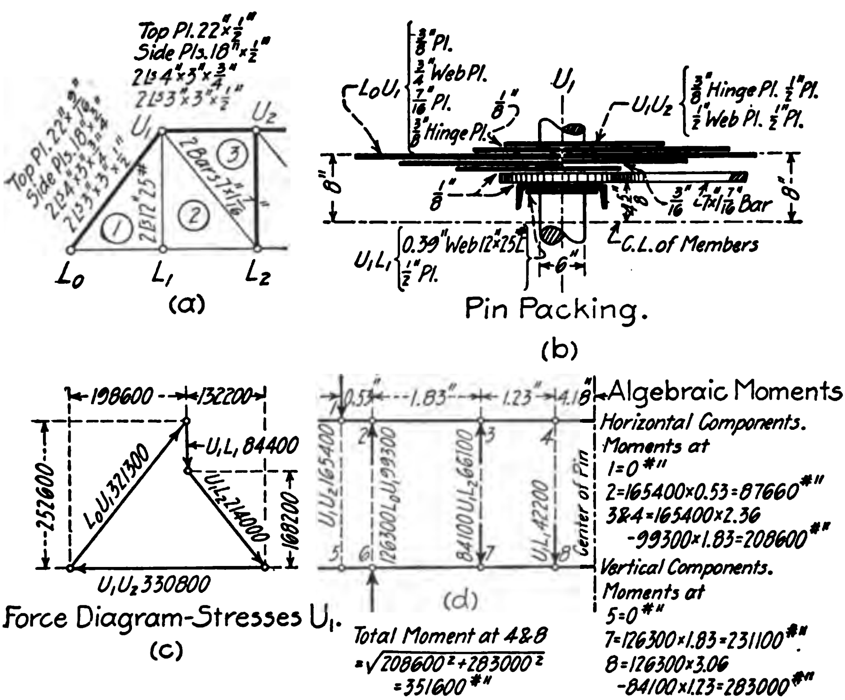



FIG. 1. STRESSES IN A PIN; ALGEBRAIC SOLUTION.

necessary to calculate the stresses when the stress in $U_1U_2$ was a maximum, and when the stress in $U_1L_2$ was a maximum. Only the case where the stress in $U_1U_2$ is a maximum will be considered. However, maximum stresses in pins sometimes occur when the stress in $U_1L_2$ is a maximum, and this case should be considered in practice.

**Bending Moment.**—The stresses in the members are shown in (c) Fig. 1, which gives the force polygon for the forces. The makeup of the members is shown in (a), and the pin packing

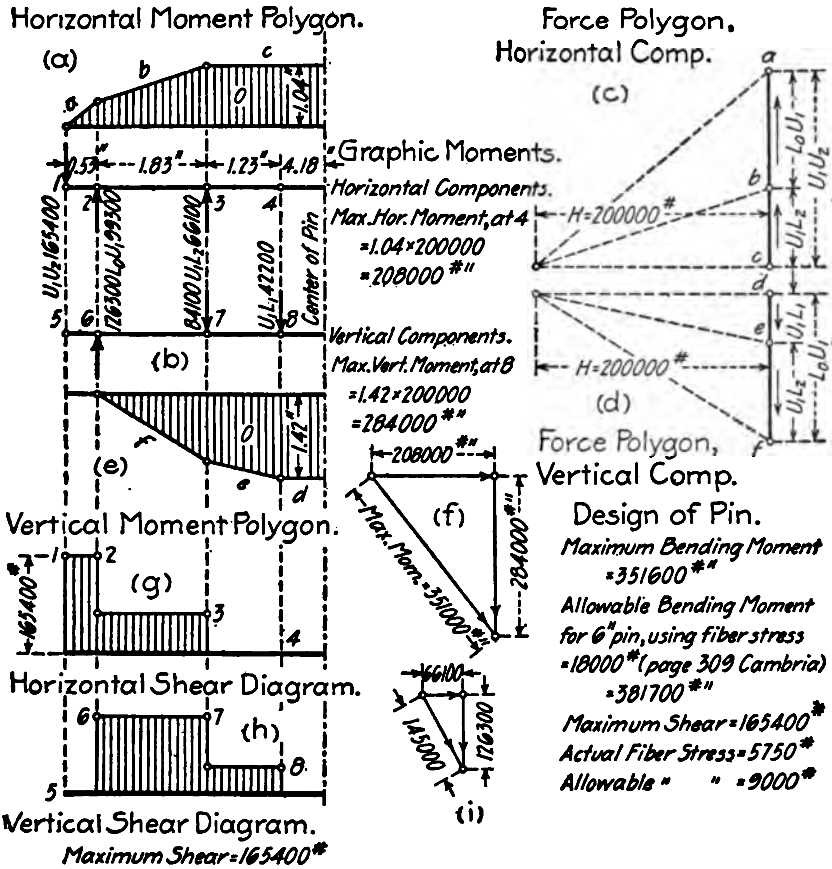



FIG. 2. STRESSES IN A PIN; GRAPHIC SOLUTION.

on one side is shown in (b). The stresses shown in (c) are applied one-half on each side of the member, the pin acting like a simple beam. The stresses are assumed as applied at the centers of the members.

*Algebraic Method.*—The amounts of the forces and the distances between their points of application as calculated from (b) are shown in (d) Fig. 1. The horizontal and vertical components of the forces are considered separately, the maximum horizontal bending moment and the maximum vertical bending moment are calculated for the same point, and the resultant moment is then found by means of the force triangle.

In (d) the horizontal bending moments are calculated about the points 1, 2, 3, 4; the maximum horizontal moment is to the right of 3, and is 208,600 in.-lb. The vertical bending moments are

calculated about points 5, 6, 7, 8; the maximum vertical bending moment is to the right of 8, and is 283,000 in.-lb. The maximum bending moment is at and to the right of 4 and 8, and is $\sqrt{208,600^2 + 283,000^2} = 351,600$ in.-lb. Substituting in the formula $S = M \cdot c/I$, the maximum bending stress is $S = 16,600$ lb. The allowable bending stress for which this bridge was designed was 18,000 lb. per square inch.

*Graphic Method.*—The amounts of the forces and the distances between their points of application are shown in (b) Fig. 2. The force polygon for the horizontal components is given in (c), and the bending moment polygon is given in (a). The maximum horizontal bending moment will be to the right of 3, and will be $H \times y = 200,000 \times 1.04 = 208,000$ in.-lb. The force polygon for the vertical forces is given in (d), and the bending moment polygon is given in (e). The maximum vertical bending moment is to the right of 8, and is $H \times y = 200,000 \times 1.42 = 284,000$ in.-lb. The maximum bending moment will occur at and to the right of 4 and 8, and will be 351,000 in.-lb., as shown in (f).

**Shear.**—The shear is found for both the horizontal and vertical components as in a simple beam, and is equal to the summation of all the forces to the left of the section. The horizontal shear diagram is shown in (g), and the vertical shear diagram is shown in (h) Fig. 2. The maximum horizontal shear is between 1 and 2, and is 165,400 lb. The shear between 2 and 3 is $165,400 - 99,300 = 66,100$ lb. The maximum vertical shear is between 6 and 7, and is 126,300 lb. The resultant shear between 2 and 3, and 6 and 7, is $\sqrt{126,300^2 + 66,100^2} = 145,000$ lb. as in (i), which is less than the horizontal shear between 1 and 2. The maximum shear, therefore, comes between 1 and 2, and is 165,400 lb. The maximum shearing unit stress is 5,750 lb. The allowable shearing stress was 9,000 lb.

**Bearing.**—The bearing stress in $L_0U_1$ is $160,650 \div 6 \times 1.94 = 13,800$ lb. Bearing stress in $U_1U_2$ is $165,400 \div 6 \times 1.88 = 14,600$ lb. Bearing stress in $U_1L_1$ is $42,200 \div 6 \times 0.89 = 7,900$ lb. Bearing stress in $U_1L_2$ is $107,000 \div 6 \times 1\frac{7}{16} = 12,400$ lb. The allowable bearing stress was 15,000 lb. per sq. in.

**COMBINED AND ECCENTRIC STRESSES.**—The combined stress due to direct and cross-bending in a tie or strut is given by the formula*

$$f = f_2 \pm f_1 = \frac{P}{A} \pm \frac{M_1 \cdot c}{I \pm \dfrac{P \cdot l^2}{k \cdot E}} \tag{1}$$

where $P$ = total direct stress in the member in lb.;
$l$ = length of the member in in.;
$I$ = moment of inertia of the member in in. to the fourth power;
$c$ = distance in in. from the neutral axis to the most remote fiber on the side for which the stress is desired;
$E$ = modulus of elasticity of the material in lb. per sq. in.;
$A$ = area of the member in sq. in.;
$f_1$ = fiber stress due to cross-bending;
$f_2 = P/A$ = direct unit stress;
$M_1$ = bending moment on the section in in.-lb.;
$k$ = a coefficient depending upon the method of loading and the condition of the ends, and is usually taken as 10 for struts with hinged ends, 24 for struts with one end hinged and the other end fixed, and 32 for both ends fixed.

The plus sign in the denominator of (1) is to be used when $P$ is a tensile stress, and the minus sign is to be used when $P$ is a compressive stress. If the member is inclined at an angle $\theta$ to the vertical, the stress $f_1$ should be multiplied by $\sin \theta$. For an eccentric stress the bending moment

* For the derivation of this formula, see "Steel Mill Buildings," Chapter XV.

is $M_1 = P \cdot e$, where $P$ is the total direct stress in the member and $e$ is the eccentricity of the load in in. (distance from the line of action of the force to the neutral axis of the member).

**Combined Compression and Cross-bending.**—The method of calculating direct and cross-bending stresses will be illustrated by calculating the stresses in the end-post of a bridge, Fig. 3, due to direct compression, weight, eccentricity of loading, and wind moment.

**End-Post.**—Design the end-post, Fig. 3, for a 160 ft. span through highway bridge. Panel length, 20′ 0″; depth of truss c. to c. of pins, 24′ 0″; length of end-post, 31′ 3″. The direct stresses are as follows: dead load stress = 30,000 lb.; live load stress = 60,000 lb.; impact =

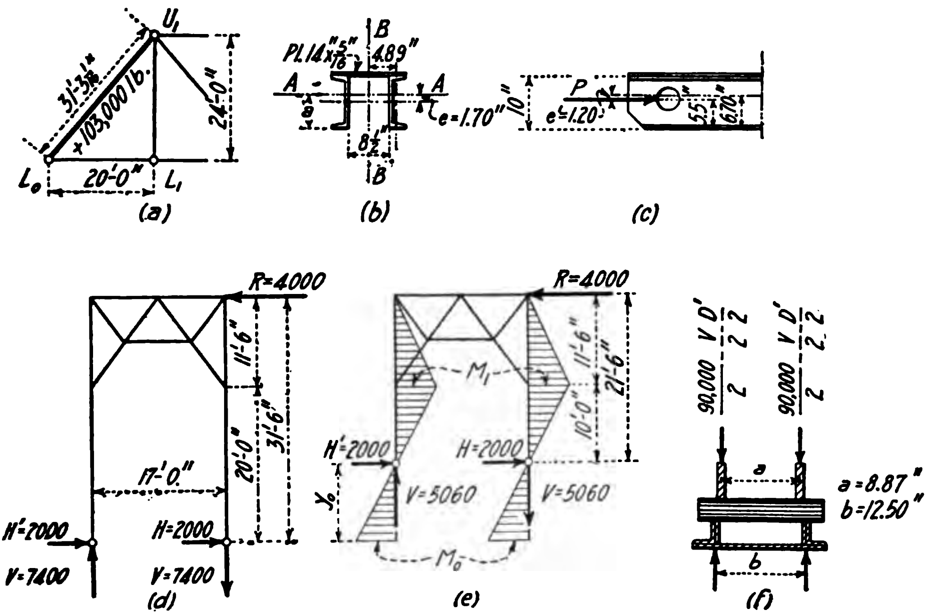



FIG. 3. END-POST OF A HIGHWAY BRIDGE.

100/(160 + 300) × 60,000 = 13,000 lb.; total direct stress due to dead load, live load and impact = 103,000 lb. The bridge is to be a class C bridge designed according to the "General Specifications for Highway Bridges," in Appendix I. From § 38 of the specifications the allowable unit stress is $f_c = 16{,}000 - 70\ l/r$. The section will be made of two channels and one cover plate. Try a section made of two 10 in. channels @ 15 lb., and one 14 in. by $\frac{5}{16}$ in. plate, (*b*), Fig. 3. From Table 17, Appendix III, the radius of gyration about horizontal axis *A–A*, is $r_A = 3.99$ in., and about the vertical axis *B–B* is, $r_B = 4.67$ in., and the eccentricity is, $e = 1.70$ in. The allowable stress is then $f_c = 16{,}000 - \dfrac{70 \times 375}{3.99} = 9{,}400$ lb. per sq. in. The required area will be $= 103{,}000 \div 9{,}400 = 10.96$ sq. in. The actual area is 13.30 sq. in. While the section appears to be excessive, it will be investigated for stress due to weight, eccentric loading and wind before rejecting it.

The area, radii of gyration and the eccentricity may be calculated as follows.

To calculate the area

| | | |
|---|---|---|
| area of two 10 in. channels | = | 8.92 sq. in. |
| area of one 14 in. by $\frac{5}{16}$ in. plate | = | 4.38 sq. in. |
| Total area | = | 13.30 sq. in |

To locate the neutral axis $A$–$A$, take moments about the lower edge of the channels

$$c = \frac{8.92 \times 5 + 4.38 \times 10.156}{13.30} = 6.70 \text{ in.}$$

The eccentricity is $e = 6.70 - 5.00 = 1.70$ in. The moment of inertia $I_A$, about axis $A$–$A$ may be calculated as follows:

Let $I_c = I$ of channels about center of channels.
$I_p = I$ of plate about center of plate.
$A_c$ = area of channels.
$A_p$ = area of plate.

Then
$$\begin{aligned} I_A &= I_c + I_p + A_c \times 1.70^2 + A_p \times 3.456^2. \\ &= 2 \times 66.9 + 0.04 + 8.92 \times 1.70^2 + 4.38 \times 3.456^2 \\ &= 133.8 + 0.04 + 25.76 + 52.20 \\ &= 211.80 \text{ in.}^4 \end{aligned}$$

Then $r_A = \sqrt{I_A \div A} = \sqrt{211.80 \div 13.3} = 3.99$ in.

The moment of inertia $I_B$, about axis $B$–$B$ may be calculated as follows.

Let $I_c' = I$ of channels about neutral axis parallel to the web.
$I_p' = I$ of plate about vertical axis.
$A_c$ = area of channels.

From Table 17, Appendix III, the distance back to back of channels is 8½ in. The distance from neutral axis to back of channel is 0.639 in. The distance from neutral axis of channels to axis $B$–$B$ is $4.25 + 0.639 = 4.889$ in. (4.89 in. will be used).

Then
$$\begin{aligned} I_B &= I_c' + I_p' + A_c \times 4.89^2 \\ &= 4.60 + 71.46 + 8.92 \times 4.89^2 \\ &= 4.60 + 71.46 + 213.28 \\ &= 289.34 \text{ in.}^4 \end{aligned}$$

Then $r_B = \sqrt{I_B \div A} = \sqrt{289.34 \div 13.3} = 4.67$ in.

*Stress Due to Weight of Member.*—The total weight of the member will be

| | |
|---|---|
| Two 10 in. channels @ 15 lb., 31′ 6″ long | = 945 lb. |
| One 14 in. × $\frac{5}{16}$ in. plate @ 14.88 lb., 30′ 0″ long | = 447 lb. |
| Details and lacing about 25 per cent | = 308 lb. |
| Total Weight, $W$ | = 1,700 lb. |

The bending moment due to weight of member is $M = \frac{1}{8} W \cdot l \cdot \sin \theta$.
Stress due to weight

$$f_w = \frac{M \cdot c}{I_A - \dfrac{P \cdot l^2}{10E}} = \frac{\frac{1}{8} W \cdot l \cdot \sin \theta \cdot c}{I_A - \dfrac{P \cdot l^2}{10E}} \qquad (2)$$

The stress due to weight in the upper fiber will be

$$f_w = \frac{\frac{1}{8} \times 1{,}700 \times 375 \times 0.645 \times 3.6125}{211.8 - \dfrac{103{,}000 \times 375^2}{10 \times 30{,}000{,}000}}$$

$$= 940 \text{ lb. per sq. in.}$$

The stress due to weight in the lower fiber is

$$\begin{aligned} f_w' &= -6.70 \times 940 \div 3.6125 \\ &= -1{,}745 \text{ lb. per sq. in.} \end{aligned}$$

*Stress Due to Eccentric Loading.*—The pins were placed ½ inch above the center of the channels, and the stress due to eccentric loading will be

$$f_e = \frac{M_1 \cdot c}{I - \dfrac{P \cdot l^2}{10E}} = \frac{P \times (1.70 - 0.5) \times c}{I - \dfrac{P \cdot l^2}{10E}} \tag{3}$$

The eccentric stress in the upper fiber will be

$$f_e = \frac{103{,}000 \times 1.20 \times 3.6125}{211.8 - \dfrac{103{,}000 \times 375^2}{10 \times 30{,}000{,}000}}$$
$$= -\ 2{,}280 \text{ lb. per. sq. in.}$$

The eccentric stress in the lower fiber is

$$f_e = +\ 6.70 \times 2{,}280 \div 3.6125$$
$$= +\ 4{,}230 \text{ lb. per sq. in.}$$

The resultant stress due to weight and eccentric loading is $f_l = f_w + f_e = +\ 940 - 2{,}280 = -\ 1{,}340$ lb. in the upper fiber, and $-\ 1{,}745 + 4{,}230 = 2{,}485$ lb. per sq. in. in the lower fiber.

The allowable stress due to weight and eccentric loading is greater than 10 per cent of the allowable stress and must be considered, with the allowable unit stress increased by 10 per cent.

The total unit stress in the member will be, $f = 103{,}000 \div 13.30 + 2{,}485 = 7{,}752 + 2{,}485 = 10{,}237$ lb. per sq. in. The allowable unit stress when weight and eccentric loading are considered is $9{,}400 \times 1.10 = 10{,}340$ lb. per sq. in., which is sufficient.

*Stress Due to Wind Moment.*—The stresses in the portal and the direct wind stresses in the end-post when the end-post is assumed as pin-connected at the base are shown in (*d*) and (*e*) Fig. 3. The end-posts may both be assumed as fixed if the windward end-post is fixed. To fix the windward end-post the bending moment must not be greater than the resisting moment which will be

$$M_0 = H \cdot y_0 = (90{,}000 - V - D')a/2$$

where $V = 5{,}060$ lb. and $D' = 7{,}000$ lb. the direct stress due to wind, and $a$ = distance center to center of metal in the sides of the end-post = 8.87 in., (*f*), Fig. 3. (The impact stress is omitted.) If $y_0$ is taken equal to $\frac{1}{2}d = 10'\ 0'' = 120$ in., we will have

$$2{,}000 \times 120 \leqq (90{,}000 - 5{,}060 - 7{,}000)8.87/2$$

which makes $240{,}000 < 345{,}600$, and the end-post may be assumed as fixed at the base. The stress due to bending moment due to wind loads in the leeward end-post will be,

$$f_w = \frac{M \cdot c}{I - \dfrac{P \cdot l^2}{10E}} \tag{4}$$
$$= \frac{240{,}000 \times 7}{289.4 - \dfrac{(90{,}000 + 5{,}060 + 7{,}000)258^2}{10 \times 30{,}000{,}000}} = 6{,}730 \text{ lb. per sq. in.}$$

The total stress due to direct wind load will be $f_w = (5{,}060 + 7{,}000)/13.30 = +\ 910$ lb. per sq. in. The total maximum wind load stress will come on the windward fiber of the leeward end-post, and will be $f_w'' = +\ 6{,}370 + 910 = +\ 7{,}280$ lb. per sq. in.

The maximum stress due to direct dead and live loads (not including impact) and wind load stresses will be

$$f = 90{,}000 \div 13.30 + 7{,}280$$
$$= 6{,}770 + 7{,}280 = 14{,}050 \text{ lb. per sq. in.}$$

From the specifications the allowable stress may be increased 50 per cent when direct and flexural wind stresses are considered.

The allowable stress when both direct and flexural wind stress are considered is then

$$f_c = 9{,}400 \times 1.50 = 14{,}100 \text{ lb. per sq. in.}$$

The stresses in the windward post will be less than in the leeward end-post calculated above.

While the section assumed appeared to be excessive, the additional area and the width of plate are required to take the flexure due to wind loads.

**Combined Tension and Cross-bending.**—The stress due to cross-bending when the member is also subjected to direct tension is given by the formula

$$f_2 = \frac{M_1 \cdot c}{I + \dfrac{P \cdot l^2}{k \cdot E}} \tag{5}$$

the nomenclature being the same as in (1). The constant $k$ is taken equal to 10 where the ends are hinged.

**Stress in a Bar Due to its Own Weight.**—Let $b$ = breadth of bar in inches; $h$ = depth of bar in inches; $w$ = weight of bar per lineal inch = 0.28 $b \cdot h$ lb.; $f_2 = P/b \cdot h$ = direct unit stress in lb. per sq. in.

We will also have $y_1 = \frac{1}{2}h$; $M_1 = \frac{1}{8}w \cdot l^2$; $P = f_2 \cdot b \cdot h$.

Substituting in (5), we have

$$f_1 = \frac{\frac{1}{8}w \cdot l^2 \cdot \frac{1}{2}h}{\dfrac{b \cdot h^3}{12} + \dfrac{f_2 \cdot b \cdot h \cdot l^2}{10 \times 28{,}000{,}000}} = \frac{4{,}900{,}000h}{f_2 + 23{,}000{,}000 \left(\dfrac{h}{l}\right)^2} \tag{6}$$

where $f_1$ is the extreme fiber stress in the bar due to weight, and is tension in lower fiber and compression in upper fiber.

If the bar is inclined, the stress obtained by formula (6) must be multiplied by the sine of the angle that the bar makes with a vertical line. Formula (6) is much more convenient for actual use than formula (5).

**Diagram for Stress in Bars Due to Their Own Weight.**—Taking the reciprocal of (6), we have

$$\frac{1}{f_1} = \frac{f_2}{4{,}900{,}000h} + \frac{23{,}000{,}000 \left(\dfrac{h}{l}\right)^2}{4{,}900{,}000h} = y_1 + y_2$$

and

$$f_1 = 1/(y_1 + y_2) \tag{7}$$

Fig. 4 gives values of $y_1$ for different values of $f_2$, and values of $y_2$ for different values of the length in feet, $L$. The values of $y_1$ and $y_2$ can be read off the diagram directly for any value of $h$, $f_2$ and $L$. And then, if the sum of $y_1$ and $y_2$ be taken on the lower part of the diagram, the reciprocal, which is the fiber stress $f_1$, may be read off the right hand side.

The use of the diagram will be illustrated by two problems:

PROBLEM I.—Required the stress in a 4 in. $\times$ 1 in. eye-bar, 20 ft. 0 in. long, which has a direct tension of 56,000 lb.

In this case, $h$ = 4 in., $L$ = 20 ft. 0 in., and $f_2$ = 14,000 lb. per sq. in. The stress due to weight, $f_1$, is found as follows: On the bottom of the diagram, Fig. 4, find $h$ = 4 inches, follow up the vertical line to its intersection with inclined line marked, $L$ = 20 feet, and then follow the horizontal line passing through the point of intersection out to the left margin and find, $y_2$ = 3.3 tens of thousandths; then follow the vertical line, $h$ = 4 inches, up to its intersection with inclined line marked, $f_2$ = 14,000, and then follow the horizontal line passing through the point of intersection out to the left margin and find, $y_1$ = 7.2 tens of thousandths.

Now to find the reciprocal of $y_1 + y_2 = 7.2 + 3.3 = 10.5$, find value of $y_1 + y_2 = 10.5$ on lower edge of diagram, follow vertical line to its intersection with inclined line marked "Line of Reciprocals" and find stress $f_1$ by following horizontal line to right hand margin to be

$$f_1 = 950 \text{ lb. per sq. in.}$$

By substituting in (6) and solving we get $f_1 = 960$ lb. per sq. in.

**Problem 2.**—Required the stress in a 5 in. × $\frac{3}{4}$ in. eye-bar, 30 ft. 0 in. long, which has a direct tension of 60,000 lb., and is inclined so that it makes an angle of 45° with a vertical line.

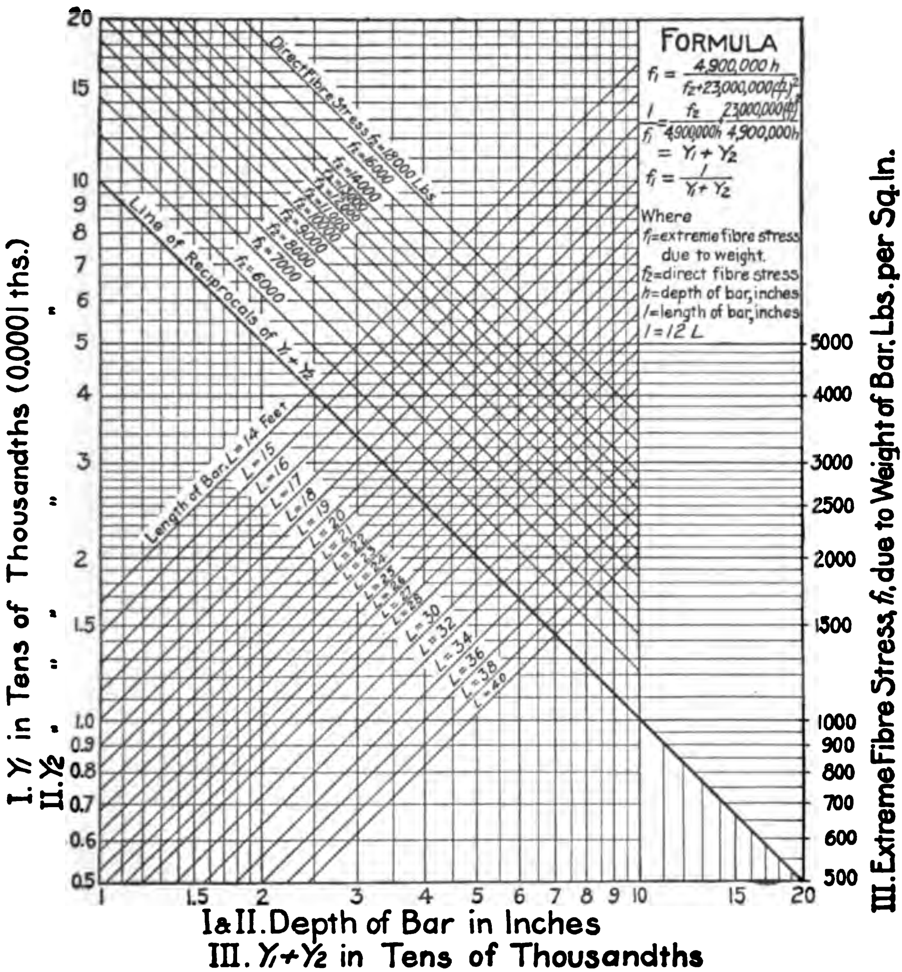



**Fig. 4. Diagram for Finding Stress in Bars Due to Their Own Weight.**

In this case, $h = 5$ in., $L = 30$ ft. 0 in., $f_2 = 16,000$ lb., and $\theta = 45°$. From the diagram, Fig. 4, as in Problem 1, $y_2 = 1.8$ tens of thousandths, and $y_1 = 6.5$ tens of thousandths, and

$$f_1 = 1/(y_1 + y_2) \times \sin\theta = 1,200 \times \sin\theta$$
$$= 850 \text{ lb. per sq. in.}$$

*Relations Between $h$, $f_1$, $f_2$ and $L$.*—For any values of $f_2$ and $L$, $f_1$ will be a maximum for that value of $h$ which will make $y_1 + y_2$ a minimum. This value of $h$ will now be determined. Differentiating equation (6) with reference to $f_1$ and $h$, we have after solving for $h$ after placing the first derivative equal to zero

$$h = L\sqrt{f_2}/4{,}800 \qquad (8)$$

in which $h$ is the depth of bar which will have a maximum fiber stress for any given values of $L$ and $f_2$.

Now if we substitute the value of $h$ in (8) back in equation (6), we find that $f_1$ will be a maximum when $y_1 = y_2$.

Now in the diagram the values of $y_1$ and $y_2$ for any given values of $f_2$ and $L$ will be equal for the depth of bar, $h$, corresponding to the intersection of the $f_2$ and $L$ lines.

It is therefore seen that every intersection of the inclined $f_2$ and $L$ lines in the diagram, has for an abscissa a value $h$, which will have a maximum fiber stress $f_1$, for the given values of $f_2$ and $L$.

For example, for $L = 30$ feet and $f_2 = 12{,}000$ lb., we find $h = 8.3$ inches and $f_1 = 1{,}700$ lb. For the given length $L$ and direct fiber stress $f_2$, a bar deeper or shallower than 8.3 inches will give a smaller value of $f_1$ than 1,700 lb.

**STRESSES IN AN ECCENTRIC RIVETED CONNECTION.**—In Fig. 5 the riveted connection carries a stress of $P = 10{,}000$ lb. The four rivets transmit a direct shear of 10,000 lb. or 2,500 lb. each, and a bending moment of $10{,}000 \times 4\frac{1}{2} = 45{,}000$ in.-lb. The shear that resists moment in each rivet acts with an arm of 2.8 in. If $R$ is the shear in each rivet due to moment, $4R \times 2.8 = 45{,}000$ in.-lb., and $R = 4{,}018$ lb.

The total shear on rivet 2, is $4{,}018 - 2{,}500 = 1{,}518$ lb.; on rivet 3, is $4{,}018 + 2{,}500 = 6{,}518$ lb.; and on rivets 1 and 4 $= \sqrt{2{,}500^2 + 4{,}018^2} = 4{,}740$ lb.

If the rivets are located at unequal distances from the center of gravity of the rivets, let $a$ represent the shear on a rivet at a unit's distance from the center of gravity; then the shear on a rivet at a distance $d_1$ from the center of gravity will be $a \cdot d_1$, and the resisting moment will be $a \cdot d_1^2$. The shear on a rivet at a distance $d_2$ from the center of gravity will be $a \cdot d_2$, and the resisting moment will be $a \cdot d_2^2$. The total resisting moment of the connection will then be $\Sigma a \cdot d^2 = M$.

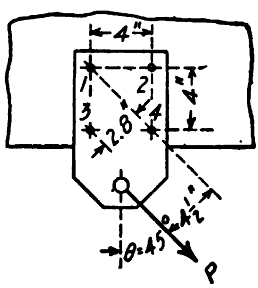



FIG. 5.

For the calculation of the stresses in the rivets of a standard riveted connection see the author's "Steel Mill Buildings," Chapter XV.

**DEFLECTION OF TRUSSES.**—When the members of a truss are stressed, the lengths of the members in compression are decreased in length, while the members in tension are increased in length. These changes in the lengths of the members cause the upper and lower chord panel points to deflect, while the positions of all other points are changed. If the left end of a bridge truss is fixed the right end will move if it is resting on free rollers. To calculate the movement of the right end of the truss proceed as follows:

In Fig. 6 the truss is fixed at $L_0$, and is free to move at $L_0'$, and is loaded with a load $W$. Under the action of the load, $L_0'$ will move a distance $\Delta$. Now assume that all the members are rigid with the exception of 1–$y$, which is increased in length the distance $\delta$, under the action of the external load $W$. The movement of the joint $L_0'$ will be $\Delta'$, and will be due to the change in length $\delta$, of the member 1–$y$. Let $H$ be the horizontal reaction necessary to bring $L_0'$ back to its

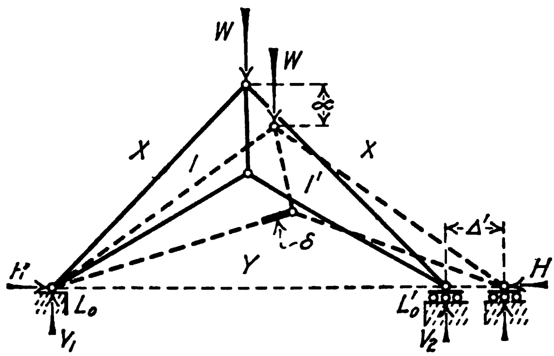



FIG. 6.

original position, and let $U \cdot H$ be the stress in the member 1–$y$ due to the horizontal thrust $H$. Now the internal work $\frac{1}{2}\delta \cdot H \cdot U$ in shortening the member 1–$y$ to its original length will be equal to the external work $\frac{1}{2}H \cdot \Delta'$, required to bring the hinge $L_0'$ back to its original position, and

$$\tfrac{1}{2}H \cdot \Delta' = \tfrac{1}{2}\delta \cdot H \cdot U$$

and

$$\Delta' = \delta \cdot U \tag{9}$$

but $\delta = P \cdot L/E$, where $P$ is the unit stress in the member 1–$y$ due to the load $W$; $L$ is the length of the member 1–$y$ in the same units as $\Delta'$; and $E$ is the modulus of elasticity of the material of the member in lb. per sq. in. Substituting this value of $\delta$ in (9) we have

$$\Delta' = P \cdot U \cdot L/E \tag{9'}$$

where $U$ is the stress in the member due to a load unity at $L_0'$ acting in the line in which $\Delta'$ is measured.

Now if each one of the remaining members of the truss is assumed as distorted in turn, the other members, meanwhile remaining rigid, the distortion at $L_0'$ will be represented by the general equation (9'), and the total deformation, $\Delta$, at $L_0'$ will be

$$\Delta = \Sigma(P \cdot U \cdot L/E) \tag{10}$$

**Algebraic Solution.**—It is required to calculate the deflection of the panel point $L_4$ in the lower chord of the 160-ft. span Pratt highway truss in Fig. 9, the stresses in the members, the areas of the members, and the lengths of the members being given in Table I. The stresses in column 4 in Table I are calculated for a full dead load and live load on the truss. In column 5 values of unit stress, $P$, are given, in column 6 values of $P \cdot L/E$ are given for $E$ = 30,000,000 lb. per sq. in. The values of $U$ in column 8 were calculated by placing a load of 1 lb. at $L_3$. The values of $P \cdot U \cdot L/E$ are given in column 9, and $\Sigma(P \cdot U \cdot L/E)$ is 1.20 in. To calculate the deflection at any other point, new values of $U$ must be calculated for a force of 1 lb. acting at the point at which the deflection is to be calculated, and acting in the direction that the deflection is to be measured.

**Graphic Solution.—Williot Deformation Diagram.**—When the deformations of the members have been calculated the relative movements of all the joints of the structure may be calculated by a graphic diagram. In (a) Fig. 7, the point $c$ is connected with points $a$ and $b$ by lines 1 and 2, respectively, which undergo changes $-\Delta_1$ and $+\Delta_2$, respectively, while the points $a$ and $b$ move to new positions $a'$ and $b'$, respectively. It is required to find the new position of the point $c$. Now if point $a$ moves to point $a'$, point $c$ will move to point $c_1$; while if point $b$ moves to point $b'$, point $c$ will move to point $c_2$. Now line 1 will be increased in length by $\Delta_1$, and line 2 will be decreased in length by $\Delta_2$. The final location of point $c$ will be at $c'$ which will be at the intersection of arcs drawn with centers $a'$ and $b'$, and radii equal to the new lengths of the lines 1 and 2,

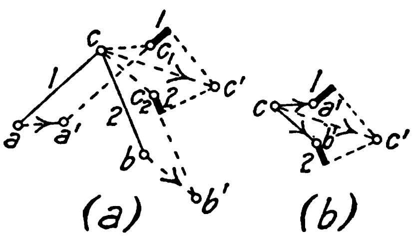


FIG. 7.

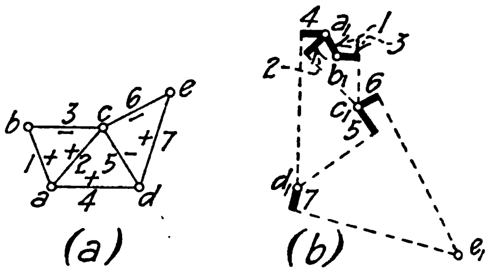


FIG. 8.

respectively. Since the deformations are very small as compared with the lengths of the members, the new location of point $c$ at the point $c'$ may be found by erecting perpendiculars at the ends of $\Delta_1$ and $\Delta_2$. The construction may be accomplished without drawing members 1 and 2, as in (b) Fig. 7. At point $c$ in (b) lay off $c$-$a'$ equal and parallel to $a$–$a'$ in (a), and also $c$–$b'$ equal and parallel to $b$–$b'$ in (a). From $a'$ lay off $\Delta_1$ away from $a'$ in (b), and from $b'$ lay off $\Delta_2$, toward $b'$ in (b). The new location of $c$ at the point $c'$ will then be found by erecting perpendiculars at the ends of the deformations $\Delta_1$ and $\Delta_2$.

With reference to signs, in Fig. 7, $-\Delta_1$ is a lengthening, and is therefore due to tension, and is called minus, while $+\Delta_2$ is a shortening, and is therefore due to compression and is called plus. In laying off the deformations in the diagram if $c$ remains fixed a tensile stress, and therefore a minus deformation requires that point $a'$ will move away from $c$, and a minus deformation should therefore be laid off on the side of the point on which the member occurs, while a plus deformation should be laid off on the side of the point opposite the side on which the member occurs. The diagram will be properly drawn if the following rule is observed in laying off the deformations: "Lay off minus deformations—tension deformations—so as to cause apparent shortening of the member, and lay off plus deformations so as to cause apparent lengthening of the member." The assumption of minus for tensile stresses and plus for compressive stresses is therefore more consistent than the opposite assumption, which is made by some writers.

In Fig. 8 the framework in (a) has deformations with signs as shown. With point $a$ assumed as a fixed point and member $a$–$b$ marked 1, assumed as a fixed axis the Williot diagram is constructed as in (b) Fig. 8. Deformation $\Delta_1$ is plus, and is laid off to cause apparent lengthening in member $a$–$b$, so that point $b_1$ will come at the lower end and point $a_1$ will come at the upper end of $\Delta_1$ in (b).

To draw the deformation diagram for the truss in Fig. 9, proceed as follows: First calculate Table I as for the algebraic method, column 7 giving the order in which the members will be used in the deformation diagram. Now begin with the member marked 1, and lay off in Fig. 9 the deformation of the two members marked $1 = +2 \times 0.078 = +0.156$ in. parallel to the member $U_2U_1U_2'$ to the prescribed scale, and mark the left end $U_2'$ and the right end $U_2$. Now lay off the deformation $2 = -0.08$ in. from $U_2$ and away from the joint $U_2$, and parallel to member

2; and lay off deformation 3 = − 0.08 in. from $U_3'$ and away from the joint $U_3'$, and parallel to member 3. Perpendiculars erected at the ends of deformations 2 and 3 will meet in the new position of $L_3$. The vertical distance between $U_3$ and $L_3$ in the diagram will be the difference in deflection of the points $U_3$ and $L_3$. At $U_3$ in the diagram lay off deformation 4 = + 0.02 in. toward the joint $U_3$ and parallel to member 4, and at $L_3$ lay off deformation 5 = − 0.121 in.

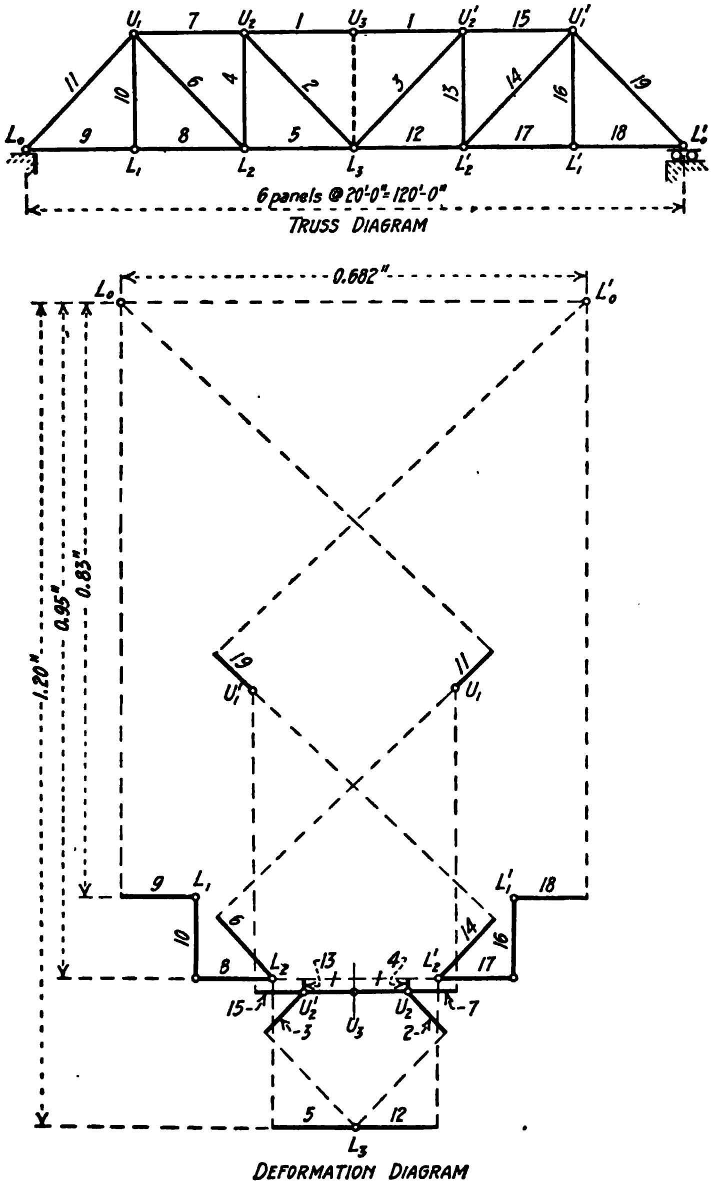



FIG. 9.

away from the joint $L_3$ and parallel to member 5. Then perpendiculars erected at the ends of deformations 4 and 5 will meet in the new position of $L_2$. In like manner perpendiculars erected at the ends of deformations 6 and 7 give the new position of point $U_1$ in the diagram, and finally perpendiculars erected at the ends of deformations 9 and 11 will give the new position of the point $L_0$. The deformation diagram for the right half of the truss is constructed in the same manner. The increase in the length of the span is 0.686 in., while the deflection of point $L_3$ below the abutments is 1.20 in., as was calculated algebraically.

## TABLE I.

### ALGEBRAIC CALCULATION OF DEFORMATIONS.

| Member. | Area in Sq. In. | Length $L$ in In. | Stress in Lb. | Unit Stress $P$ in Lb. | $\frac{PL}{E}$ | No. Mem. | $U$ | $\frac{PUL}{E}$ |
|---|---|---|---|---|---|---|---|---|
| $U_1U_2$ | 8.70 | 240 | + 75,600 | + 8,700 | + 0.070 | 7 | + 0.96 | + 0.067 |
| $U_2U_3$ | 8.70 | 240 | + 85,100 | + 9,800 | + 0.078 | 1 | + 1.44 | + 0.112 |
| $L_0L_1$ | 3.44 | 240 | − 47,300 | − 13,800 | − 0.110 | 9 | − 0.48 | + 0.053 |
| $L_1L_2$ | 3.44 | 240 | − 47,300 | − 13,800 | − 0.110 | 8 | − 0.48 | + 0.053 |
| $L_2L_3$ | 5.00 | 240 | − 75,600 | − 15,100 | − 0.121 | 5 | − 0.96 | + 0.116 |
| $L_0U_1$ | 10.70 | 348 | + 68,650 | + 6,400 | + 0.074 | 11 | + 0.69 | + 0.051 |
| $U_1L_1$ | 1.44 | 252 | − 19,900 | − 14,000 | − 0.12 | 10 | 0.0 | 0.0 |
| $U_1L_2$ | 3.13 | 348 | − 31,200 | − 10,000 | − 0.12 | 6 | − 0.69 | + 0.083 |
| $U_2L_2$ | 3.90 | 252 | + 9,450 | + 2,400 | + 0.020 | 4 | + 0.50 | + 0.010 |
| $U_2L_3$ | 2.00 | 348 | − 13,700 | − 6,900 | − 0.080 | 2 | − 0.69 | + 0.055 |
| $U_3L_3$ | 3.90 | 252 | 0 | 0 | 0.0 | | 0.0 | 0.0 |
| Total deformation = + 0.600 × 2 = 1.200 in. | | | | | | | | + 0.600 |

The graphic method gives the relative positions of all points in the truss, while the algebraic method gives the deflection of one point, only.

For the deformation diagrams of trusses unsymmetrically loaded, and for the methods of calculating the stresses in two-hinged arches see the author's "Steel Mill Buildings," Chapter XIV.

**STRESSES IN ROLLERS.**—When a cylindrical roller is pressed between two plates the roller is deformed so that the linear element of the roller in contact is spread out as the pressure increases. It has been found by experiment that the plates are but little deformed in comparison with the deformation of the rollers for stresses within the elastic limit, so that the entire deformation may be considered as occurring in the rollers. (For a more complete discussion see Merriman's "Mechanics of Materials.")

In (b) Fig. 10 the vertical diameter $A–A$ is shortened to $B–B$, and the shortening in a half diameter is $A–B = e$; also let $y$ be the shortening in any half chord. Now if the stress at $B$ is $S$, and at the point whose coördinates are $x$, $y$ is $S'$, then if the elastic limit of the material is not exceeded

$$S/S' = e/y, \text{ or } S' = S \cdot y/e$$

Now $2e/d$ is the unit shortening in the vertical diameter, and this is equal to $S/E$, and

$$S/E = 2e/d \tag{11}$$

Now the stress $S'$ acts over the entire area $l \cdot dx$, and

$$\int S' \cdot l \cdot dx = W \tag{12}$$

where $W$ is the total stress on the rollers. Equations (11) and (12) are the equations for finding $S$ and $e$. Substituting $S' = S \cdot y/e$ in (12) we have

$$S \cdot l \int y \cdot dx = W \cdot e \tag{13}$$

Now $\int y \cdot dx$ = the area of the segment compressed, which may be considered a parabola $= \frac{2}{3}$ chord $1\text{–}2 \cdot e$. Now chord $1\text{–}2 = 2(2 \cdot d)^{\frac{1}{2}}$, approximately, and (13) becomes

$$S \cdot l \tfrac{4}{3} e(e \cdot d)^{\frac{1}{2}} = W \cdot e \tag{14}$$

Solving equations (11) and (14), we have

$$W = \tfrac{2}{3} l \cdot d \cdot S(2S/E)^{\frac{1}{2}} \tag{15}$$

or

$$w = \tfrac{2}{3} d \cdot S(2S/E)^{\frac{1}{2}} \tag{16}$$

where $w$ is the load per lineal inch of roller, if $d$ is given in inches.

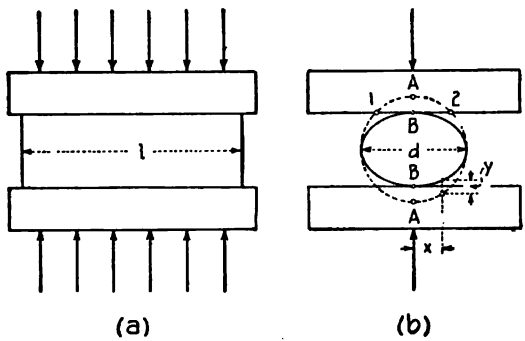



FIG. 10.

Now taking $S = 24{,}000$ lb. per sq. in., and $E = 30{,}000{,}000$ lb. per sq. in., equation (16) becomes

$$w = 640d \tag{17}$$

**CAMBER.**—Bridges are constructed so that when loaded the trusses will take the form assumed in the calculations. This may be done in two ways: (1) by increasing the lengths of all compression members and decreasing the lengths of all tension members the amounts calculated as in column 6 in Table I—this method requires laborious calculations and increases the labor in making and checking the drawings; (2) the most common method is to increase the lengths of the top chords $\frac{1}{4}$ in. in 10 ft. for railway bridges and $\frac{3}{16}$ in. in 10 ft. for highway bridges over the lengths of the lower chords. This method is very easy to apply and satisfies theoretical requirements quite closely.

Let $c$ = camber in inches at the center of the span;
$a$ = total increase in the length of the top chord required to produce the camber, in inches;
$h$ = the height of the truss in feet;
$l$ = the length of span in feet;

then

$$c = a \cdot l/8h, \text{ and } a = 8c \cdot h/l \tag{18}$$

Now in the 160-ft. span in Fig. 9, $c = 1.20$ in., $l = 160$ ft. 0 in., and $h = 21$ ft. 0 in. Then

$$a = 8 \times 1.20 \times 21/160 = 1.26 \text{ inches.}$$

Now this increase will be put in 4 panels, giving 0.32 in. in each panel, or 0.16 in. in 10 ft. This is slightly less than $\frac{3}{16}$ in. for each 10 ft. as commonly specified for highway bridges.

# CHAPTER VII.

## THE SOLUTIONS OF PROBLEMS IN THE CALCULATION OF STRESSES IN BRIDGE TRUSSES.

**Introduction.**—To obtain a thorough knowledge of the calculation of stresses in trusses it is necessary to solve numerous problems. The problems in this chapter have been selected with care and have shown their value by actual use in the class-room. A problem is first solved and the solution is followed through in detail. A second problem of a similar character is then stated and left for the student to solve. These problems should be solved in connection with the study of the preceding chapters.

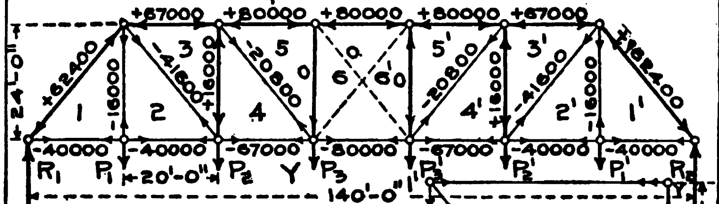
**Instructions.**—(1) *Plate*: The standard plate is to be 9″ × 10½″, with a 1″ border on the left-hand side and a ½″ border on the top, bottom and right-hand side of the plate. The plate inside the border is to be 7½″ × 9½″. (2) *Coördinates*: Unless stated to the contrary, the coördinates given in the data will refer to the lower left-hand corner of the plate as the origin of co-ordinates. (3) *Data*: Complete data shall be placed on each problem so that the solution will be self-explanatory. The span, panel length, depth, roadway and other dimensions shall be shown on the truss diagram and shall be stated in a prominent place. The loads shall be stated, and the values of all trigonometric functions shall be given to three decimal places. (4) *Lettering*: All lettering shall be in Engineering News style. The main headings shall be made with capitals 0.2″ high, and lower case letters ⅗ of this height. Capitals in the body of the problem are to be 0.15″ in height, and the lower case letters are to be ⅗ of this height. (5) *Scales*: The scale of the forces and of the trusses shall be given as 1″ = (——) lb., or ft.; and by a graphic scale as well. (6) *Name*: The name of the student is to be placed outside the border in the lower right-hand corner. (7) *Equations*: All equations shall be given, but details of the solution may be indicated. (8) *References*: References are to "The Design of Highway Bridges."

NOTE.—It should be noted that all the problems have been reduced so that all dimensions are one-half the original dimensions given in the statements of the problems.

## Bridge Analysis. Problem 2.

### Pratt Truss.

### X Graphic Resolution. X




Dead Load = 800 # per lin. ft. per truss.

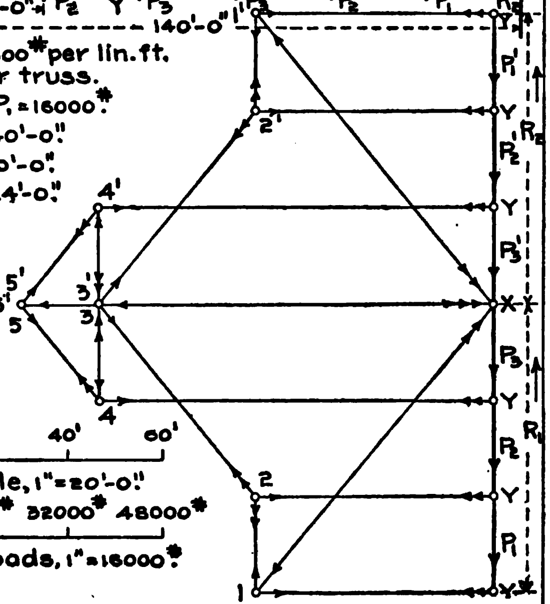
Joint Load, P, = 16000 #.

Span, L, = 140'-0".

Panel, l, = 20'-0".

Depth, d, = 24'-0".

N = 7.




0' 20' 40' 60'

Truss Scale, 1" = 20'-0".

0 # 16000 # 32000 # 48000 #

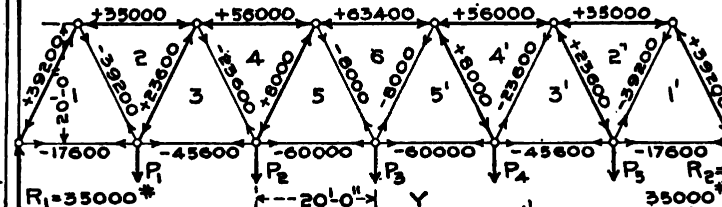
Scale of Loads, 1" = 16000 #.

Dead Load Stress Diagram.

## Bridge Analysis. Problem 1.

### X Warren Truss. X

### Graphic Resolution.




Dead Load 700 # per lineal foot per truss.

0' 16'

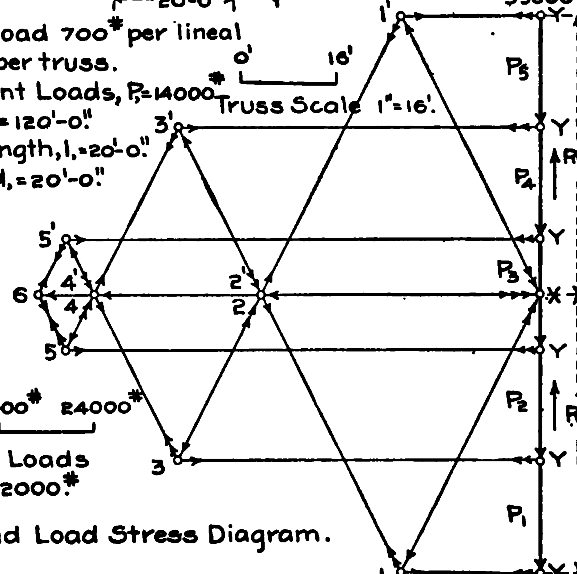
Truss Scale 1" = 16'.

Equal Joint Loads, P, = 14000 #

Span, L, = 120'-0".

Panel length, l, = 20'-0".

Depth, d, = 20'-0".




0 # 12000 # 24000 #

Scale of Loads 1" = 12000 #.

Dead Load Stress Diagram.

### Problem 1. Dead Load Stresses in a Warren Truss by Graphic Resolution.

(*a*) **Problem.**—Given a Warren truss, span 120′ 0″, panel length 20′ 0″, depth 20′ 0″, dead load 700 lb. per ft. per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 16′ 0″. Scale of loads, 1″ = 12,000 lb.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. The calculation of the stresses is started at the left reaction, and the stress diagram is closed at the right reaction. For additional information on the solution see Chapter III.

(*c*) **Results.**—The top chord is in compression, the bottom chord is in tension; all web members leaning toward the center of the truss are in compression, while the web members leaning toward the abutments are in tension. All web members meeting on the unloaded chord (top chord) have stresses equal in amount but opposite in sign. The stresses in the lower chord are the arithmetical means of the stresses in adjacent panels of the top chord. Warren trusses are commonly made of iron or steel with riveted connections, the most common section being two angles placed back to back.

### Problem 1A. Dead Load Stresses in a Warren Truss by Graphic Resolution.

(*a*) **Problem.**—Given a Warren truss, span 140′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 600 lb. per ft. per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 20′ 0″. Scale of loads, 1″ = 12,000 lb.

### Problem 2. Dead Load Stresses in a Pratt Truss by Graphic Resolution.

(*a*) **Problem.**—Given a Pratt truss, span 140′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 800 lb. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1′ = 20′ 0″. Scale of loads, 1″ = 16,000 lb.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate the stresses by graphic resolution, beginning at $R_1$ and checking up at $R_2$, following the order shown in the stress diagram.

(*c*) **Results.**—The top chord is in compression and the bottom chord is in tension as in the Warren truss. The inclined members are in tension, while the vertical members are in compression. Member 1–2 is simply a hanger. There is no stress due to dead loads in the diagonal members in the middle panel of a Pratt truss with an odd number of panels. The stresses in the posts are equal to the inclined components of the stresses in the inclined members, meeting them on the unloaded chord (top chord). Stresses in certain panels in the top and bottom chord are equal. The Pratt truss is quite generally used for steel bridges and is also used for combination bridges, where the tension members are made of iron or steel and the compression members are made of timber.

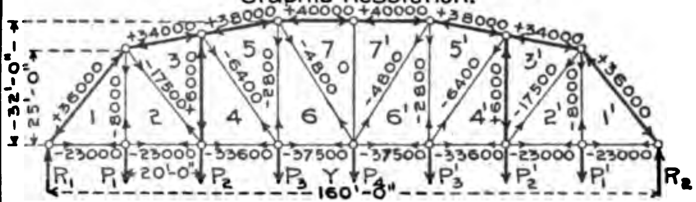
### Problem 2A. Dead Load Stresses in a Pratt Truss by Graphic Resolution.

(*a*) **Problem.**—Given a Pratt truss, span 160′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 800 lb. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 20,000 lb.

## Bridge Analysis. Problem 4.

### Camel Back Truss.

### Graphic Resolution.




Dead Load = 400# per lin. ft. per truss.

Joint Load, $P_1$ = 8000#.

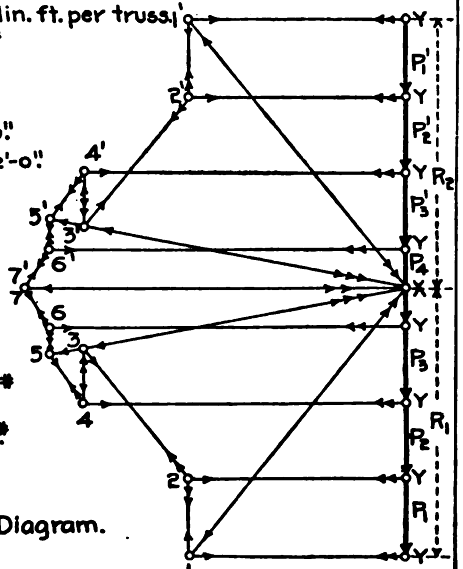
Span, L, = 160'-0".

Panel, l, = 20'-0".

Depth, $d_1$, Hip, = 25'-0".

Depth, $d_2$, Center, = 32'-0".

N = 8.



0' 25' 50'

Truss Scale, 1" = 25'-0".

0# 10000# 20000#

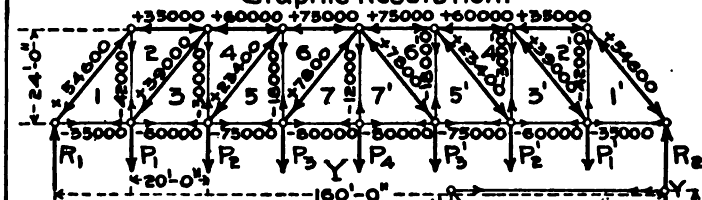
Load Scale, 1" = 10000#.

Dead Load Stress Diagram.

## Bridge Analysis. Problem 3.

### Howe Truss.

### Graphic Resolution.




Dead Load = 600# per lineal ft. per truss.

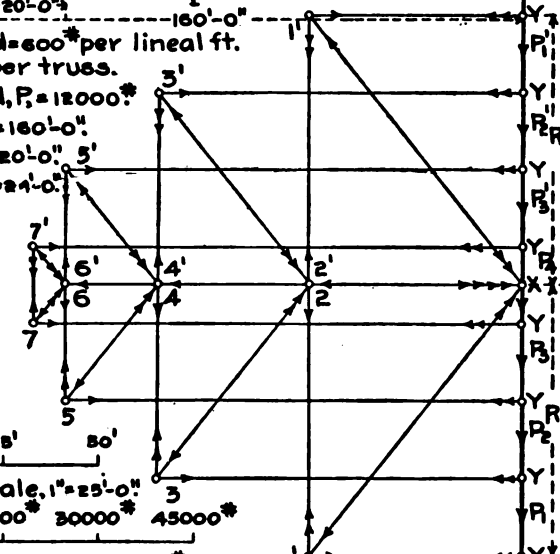
Joint Load, $P_1$ = 12000#.

Span, L, = 160'-0".

Panel, l, = 20'-0".

Depth, d, = 24'-0".

N = 8.



0' 25' 50'

Truss Scale, 1" = 25'-0".

0# 15000# 30000# 45000#

Scale of Loads, 1" = 15000#.

Dead Load Stress Diagram.

### Problem 3. Dead Load Stresses in a Howe Truss by Graphic Resolution.

(*a*) **Problem.**—Given a Howe truss, span 160′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 600 lb. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 15,000 lb.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate the stresses by graphic resolution, beginning at $R_1$ and checking at $R_2$, following the order shown in the stress diagram.

(*c*) **Results.**—The top chord is in compression and the bottom chord is in tension as in the Warren truss. All inclined members are in compression, while all vertical members are in tension. The stresses in the verticals are equal to the vertical components of the stresses in the diagonal members meeting them on the unloaded chord. Stresses in certain panels in the top and bottom chord are equal.

The Howe truss when used for highway or railroad bridges is commonly built with timber top and bottom chords and timber diagonal struts, the only iron being the vertical ties and the cast iron angle blocks to take the bearing of the timber struts. This makes a very satisfactory truss and is quite economical where timber is cheap.

### Problem 3a. Dead Load Stresses in a Howe Truss by Graphic Resolution.

(*a*) **Problem.**—Given a Howe truss, span 162′ 0″, panel length 18′ 0″, depth 24′ 0″, dead load 600 lb. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 15,000 lb.

### Problem 4. Dead Load Stresses in a Camel-back Truss by Graphic Resolution.

(*a*) **Problem.**—Given a Camel-back (inclined Pratt) truss, span 160′ 0″, panel length 20′ 0″ depth at the hip 25′ 0″, depth at the center 32′ 0″, dead load 400 lb. per lineal foot per truss, Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 10,000 lb.

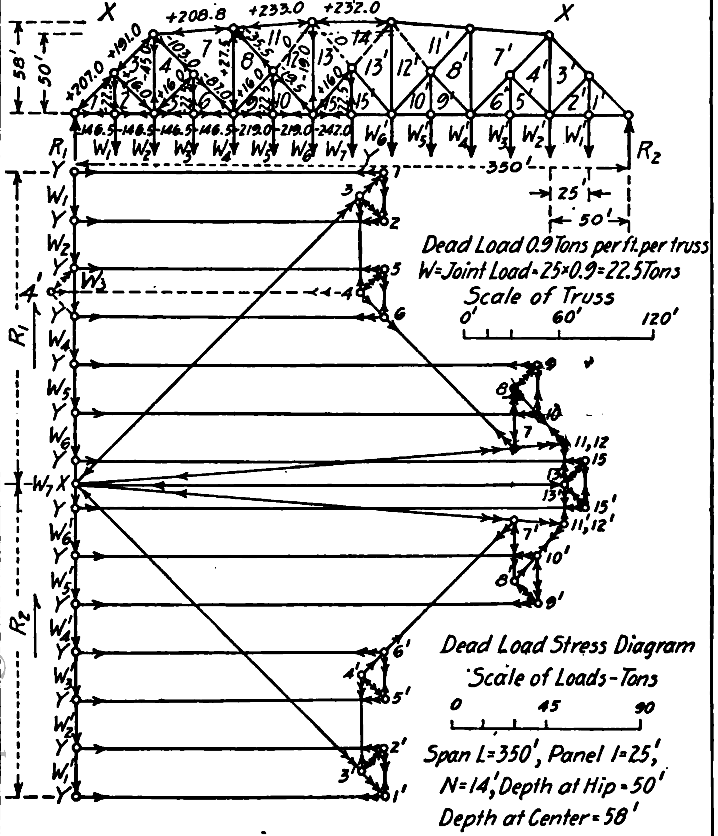
(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate the stresses by graphic resolution, beginning at $R_1$ and checking up at $R_2$. Follow the order given in the stress diagram.

(*c*) **Results.**—The top chord is in compression and the bottom chord is in tension the same as in the Pratt truss. All inclined web members are in tension; while part of the posts are in compression and part are in tension. Member 1–2 is simply a hanger and is always in tension. This type of truss is quite generally used for steel and combination bridges for spans from 150 to 200 feet, and also for long span roof trusses. In the roof truss, the loads are on both the top and bottom chords or on the top chord alone.

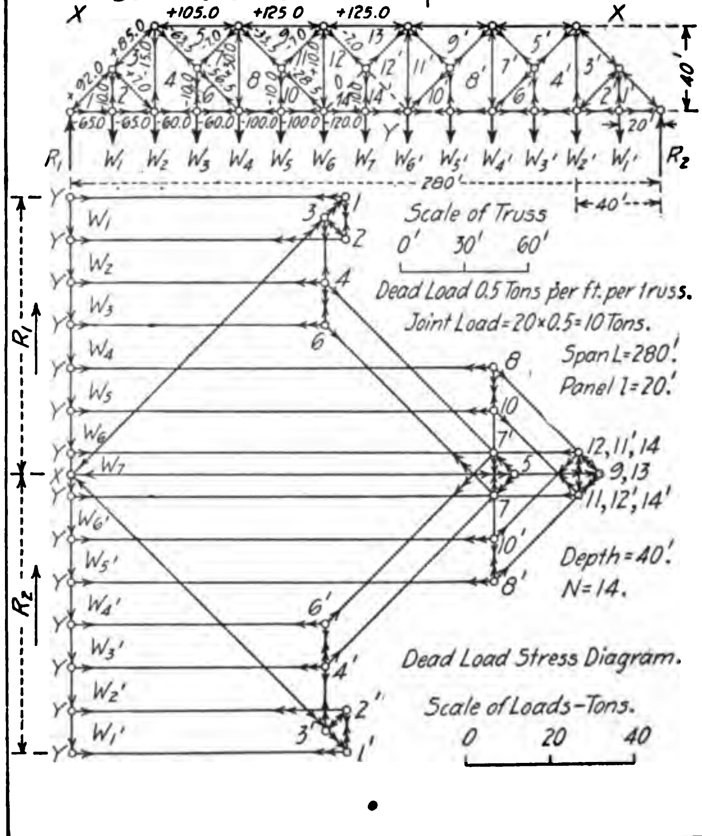
### Problem 4a. Dead Load Stresses in a Camel-back Truss by Graphic Resolution.

(*a*) **Problem.**—Given a Camel-back (inclined Pratt) truss, span 180′ 0″, panel length 20′ 0″ (three panels with parallel chords), depth at the hip 25′ 0″, depth at the center 32′ 0″, dead load 400 lb. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 12,000 lb.

## Bridge Analysis. Problem 6.
### Petit Truss—Graphic Resolution.




## Bridge Analysis Problem 5.
### Baltimore Truss — Graphic Resolution.



### PROBLEM 5. DEAD LOAD STRESSES IN A BALTIMORE TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Baltimore truss, span 280′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 0.5 tons per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 40′ 0″. Scale of loads, 1″ = 40 tons.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the top downwards. Calculate $R_1$ and $R_2$. Calculate the stresses at the left reaction by constructing triangle $1-Y-X$ as shown. Then calculate the stress in 1–2 by constructing polygon $Y-1-2-Y$. Draw 3–2, which is the stress in member 3–2. Then calculate the stress in 3–4 and $4-Y$ by constructing polygon $Y-2-3-4-Y$. Calculate the stresses in the remaining members in order, finally checking up at $R_2$.

(*c*) **Results.**—It will be see that the Baltimore truss is a Pratt truss with subdivided panels· The stresses in the first and second panels of the lower chord are larger than the stresses in the third and fourth panels of the lower chord. The stress in 6–7 is equal to the inclined component of the shear in the panel, plus the stress due to the half load that is carried toward the center of the bridge by 5–7. The Baltimore truss is used for long spans in which short panels can be used with an economical depth.

### PROBLEM 5A. DEAD LOAD STRESSES IN A BALTIMORE TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Baltimore truss, span 320′ 0″, panel length 20′ 0″, depth 50′ 0″, dead load 0.5 tons per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 50′ 0″. Scale of loads, 1″ = 50 tons.

### PROBLEM 6. DEAD LOAD STRESSES IN A PETIT TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Petit truss, span 350′ 0″, panel length 25′ 0″, depth at hip 50′ 0″ depth at center 58′ 0″, dead load 0.9 tons per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss, 1″ = 50′ 0″. Scale of loads, 1″ = 45 tons.

(*b*) **Methods.**—The loads beginning with the first load on the left are laid off from the top downwards. Calculate $R_1$ and $R_2$. Calculate the stresses in the members at the left reaction by constructing force triangle $1-Y-X$. Then calculate the stress in 1–2 by constructing polygon $Y-1-2-Y$. Draw 3–2, which is the stress in member 3–2. Then pass to joint $W_2$ where there appears to be an ambiguity, stress 4–5 being unknown. To remove the ambiguity proceed as follows: At $W_2$ on the left side of the stress diagram assume that $W_2$ is the stress in 5–6 (the member 5–6 is simply a hanger and the stress is as assumed). Calculate the stress in 4–5 by completing the triangle of stresses in the auxiliary members. The stresses are now all known at $W_2$ except 3–4 and $5-Y$, but the stress in 4–5 is between the two unknown stresses. First complete the force polygon $2-3-4-5'-Y-Y-2$. Then by changing the order the true polygon $2-3-4-5-Y-Y-2$ may be drawn. This solution is sometimes called the method of sliding in a member. The apparent ambiguity at joint $W_4$ may be removed in the same manner. The stress diagram is carried through as shown and finally checked up at $R_2$. It will be seen that there is no apparent ambiguity on the right side of the truss.

(*c*) **Results.**—It will be seen that the Petit truss is an inclined Pratt or Camel-back truss with subdivided panels. The auxiliary members are commonly tension members in all except the end primary panels as in the Baltimore truss in Problem 5. It will be seen that the stresses in the first four panels of the lower chord are the same. The loads in this type of Petit truss are carried directly to the abutments. The Petit truss is quite generally used for long span highway and railway bridges.

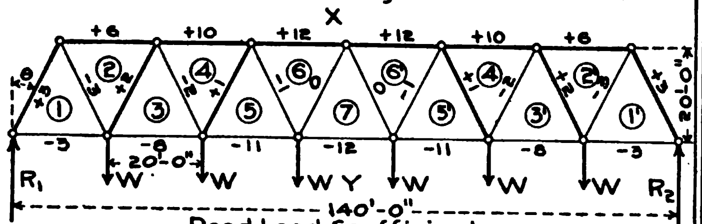
### PROBLEM 6A. DEAD LOAD STRESSES IN A PETIT TRUSS BY GRAPHIC RESOLUTION.

(*a*) **Problem.**—Given a Petit truss with the same span, panel length, depths, and dead load as in Problem 6; the auxiliary members being arranged as in the Baltimore truss in Problem 5.

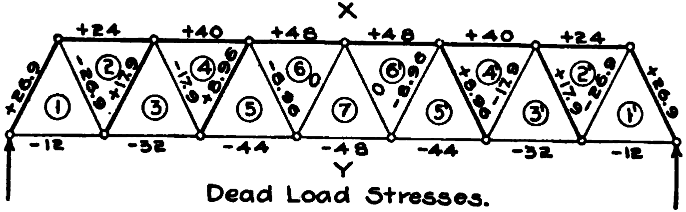
## Bridge Analysis. Problem 8.

### Warren Truss.

### Dead Load Stresses. Algebraic Resolution.




Dead Load Coefficients.




Dead Load Stresses.

Joint Load, W, = 8 tons.

Chord Stresses = Coefficient x W x tan θ.

Web " = Coefficient x W x sec θ.

Span, L, = 140'-0". Sec θ = 1.12.

Panel length, l, = 20'-0". Tan θ = 0.50.

Depth, d, = 20'-0".

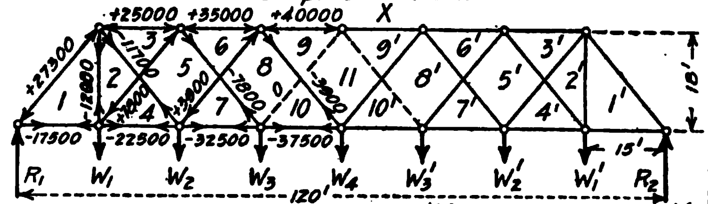
Stresses in Chords = Coeff. x W tan θ

" " Webs = " x W sec θ

## Bridge Analysis. Problem 7.

### Quadrangular Warren Truss

*Graphic Resolution*




Y

*Bending Moment at center*

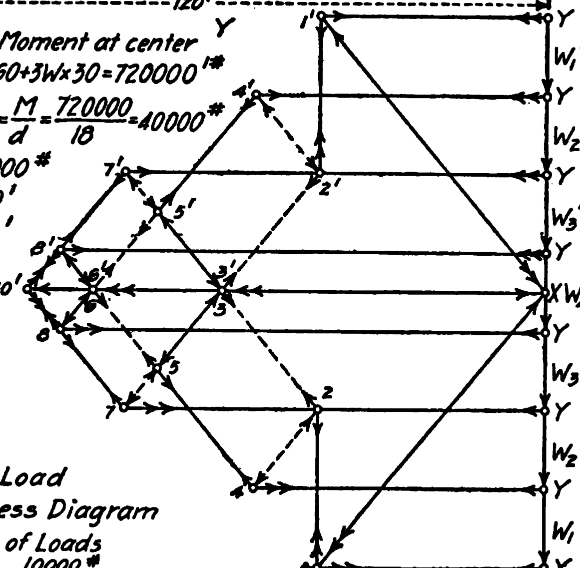
$= -R_1 \times 60 + 3W \times 30 = 720000$ '#

*Stress* 9-X $= \frac{M}{d} = \frac{720000}{18} = 40000$#

*Load W = 6000*#

*Span = 120'*

*Depth = 18'*




*Dead Load*

*Stress Diagram*

*Scale of Loads*

*0# 10000#*

### PROBLEM 7. DEAD LOAD STRESSES IN A QUADRANGULAR WARREN TRUSS BY GRAPHIC RESOLUTION.

(a) **Problem.**—Given a quadrangular Warren truss, span 120′ 0″, panel length 15′ 0″, depth 18′ 0″, dead load 400 lb. per lineal foot per truss. Calculate the dead load stresses by graphic resolution. Scale of truss and loads as shown.

(b) **Methods.**—The loads beginning with the first load on the left are laid off from the bottom upwards. Calculate $R_1$ and $R_2$. The stresses at $R_1$ may be calculated by constructing force polygon 1–$X$–$Y$. However, on passing to the next joint in either the top or bottom chord the solution is indeterminate. To solve the problem calculate the stress in the top chord 9–$X$ by taking moments about the center joint in the bottom chord, the stress in 9–11 being zero. Lay off 9–$X$ in the stress diagram and complete the diagram to the left and the right of the center as shown. It will be seen that the stresses in certain members occur twice in the diagram. The truss diagram can be divided into two systems as in Problem 14, and the stresses can be calculated for each system, the chord stresses in the two systems being added together for the final stresses.

(c) **Results.**—The quadrangular Warren truss is a double intersection truss in which the stresses are statically determinate for dead loads but are statically indeterminate for live load web stresses, as will be shown in Problem 14. This truss, built with riveted connections, is extensively used by the American Bridge Company for highway bridges for spans from 80 to 152 feet.

### PROBLEM 7A. DEAD LOAD STRESSES IN A QUADRANGULAR WARREN TRUSS BY GRAPHIC RESOLUTION.

(a) **Problem.**—Given a quadrangular Warren truss, span 150′ 0″, panel length 15′ 0″, depth 18′ 0″, dead load 500 lb. per lineal foot per truss. Calculate the dead and live load stresses by graphic resolution. Scale of truss, 1″ = 20′ 0″. Scale of loads, 1″ = 10,000 lb.

### PROBLEM 8. DEAD LOAD STRESSES IN A WARREN TRUSS BY ALGEBRAIC RESOLUTION (METHOD OF COEFFICIENTS).

(a) **Problem.**—Given a Warren truss, span 140′ 0″, panel length 20′ 0″, depth 20′ 0″, dead load 800 lb. per lineal foot per truss. Calculate the dead load stresses by algebraic resolution. Scale of truss, 1″ = 20′ 0″.

(b) **Methods.**—Beginning at the left the left reaction $R_1 = 3W$. The shear in the first panel is $3W$, in the second panel is $2W$, in the third panel is $W$, and in the fourth panel is zero. Now resolving at $R_1$ the stress in 1–$Y$ $= -3W\cdot\tan\theta$, stress 1–$X$ $= +3W\cdot\sec\theta$. Cut members 1–$Y$, 1–2 and 2–$X$ and the truss to the right by a plane and equate the horizontal components of the stresses in the members. The unknown stress 2–$X$ will equal the sum of the horizontal components of the stresses in 1–$Y$ and 1–2 with sign changed, $= -(-3-3)W\cdot\tan\theta = +6W\cdot\tan\theta$. The stress in 3–$Y$ $= -(6+2)W\cdot\tan\theta = -8W\cdot\tan\theta$. Stress in 4–$X$ $= -(-8-2)W\cdot\tan\theta = +10W\cdot\tan\theta$; stress in 5–$Y$ $= -(+10+1)W\cdot\tan\theta = +11W\cdot\tan\theta$; and the stress in 6–$X$ $= -(-11-1)W\cdot\tan\theta = +12W\cdot\tan\theta$. The coefficients of the chord stresses when multiplied by $W\cdot\tan\theta$ give the stresses, while the coefficients for the webs when multiplied by $W\cdot\sec\theta$ give the web stresses.

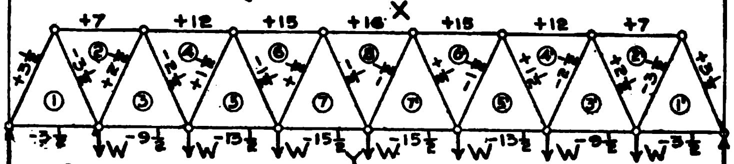
(c) **Results.**—In the method of coefficients the shears are calculated first, and the chord coefficients follow easily by summing the horizontal components. This method is the shortest and the best for the calculation of the stresses in bridge trusses with parallel chords.

### PROBLEM 8A. DEAD LOAD STRESSES IN A WARREN TRUSS BY ALGEBRAIC RESOLUTION.

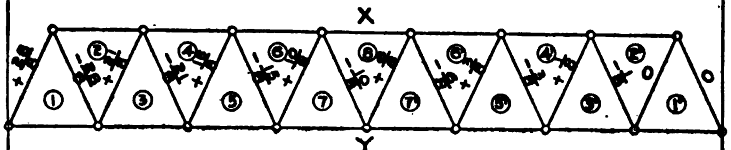
(a) **Problem.**—Given a Warren truss, span 160′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 700 lb. per lineal foot per truss. Calculate the dead load stresses by algebraic resolution. Scale of truss, 1″ = 25′ 0″.

## Bridge Analysis. Problem 10.

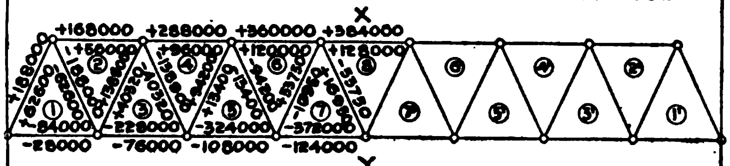
### Maximum and Minimum Stresses in a Warren Truss. Algebraic Resolution.




Dead Load Coefficients and Maximum Chord Coefficients.




Max. Live Load Web Coef. Min. Live Load Web Coef.




Max. and Min. Stresses for Dead and Live Loads.

Dead Load, 800# per foot per truss. W = 16000#.

Live Load, 1600# " " " " . P = 32000#.

Span, L, = 160'-0". Sec θ = 1.12.

Panel, l, = 20'-0". Tan θ = 0.50.

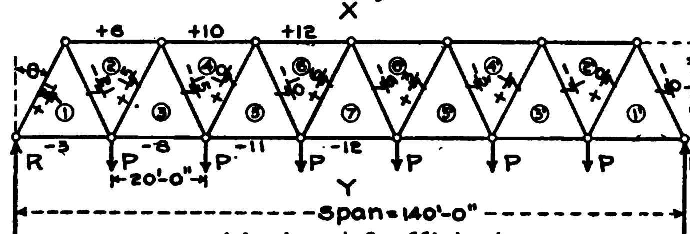
Depth, d, = 20'-0".



## Bridge Analysis. Problem 9.

### Warren Truss.

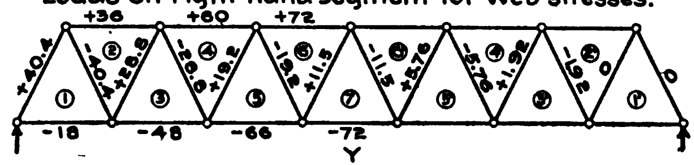
### Live Load Stresses. Algebraic Resolution.




Live Load Coefficients.

All loads on for maximum chord stresses.

Loads on right hand segment for web stresses.




Maximum stresses. Minimum web stresses.

Joint Load, P, = 12 tons.

Chord stresses = Coefficient × P × tan θ.

Web stresses = Coefficient × P × sec θ.

Span, L, = 140'-0". Sec θ = 1.12.

Panel, l, = 20'-0". Tan θ = 0.50.

Depth, d, = 20'-0".

Stresses in Chords = Coeff. × P tan θ

" " Webs = " × P sec θ

PROBLEM 9. LIVE LOAD STRESSES IN A WARREN TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Warren truss, span 140′ 0″, panel length 20′ 0″, depth 20′ 0″, live load 1,200 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to the live load by algebraic resolution. Scale of truss, 1″ = 20′ 0″.

(*b*) **Methods.**—Construct two truss diagrams as shown.

*Chord Stresses.*—The maximum chord stresses occur when the joints are all loaded, and the chord coefficients are found as in Problem 8. The minimum live load stresses in the chords occur when none of the joints are loaded, and are zero for each member.

*Web Stresses.*—The maximum web stresses in any panel occur when the longer segment into which the panel divides the truss is loaded, while the shorter segment has no loads on it. The minimum live load web stresses occur when the shorter segment is loaded and the longer segment has no loads on it. The maximum stresses in members 1–$X$ and 1–2 occur when the truss is fully loaded. The shear in the panel is $3P$, or $21/7P$, and the stress in 1–$X$ = $3P \cdot \sec\theta = +40.4$ tons, while the stress in 1–2 = $-3P \cdot \sec\theta = -40.4$ tons. The minimum stresses in 1–$X$ and 1–2 are zero. The maximum stresses in 2–3 and 3–4 occur when 5 loads are on the right of the panel and there are no loads on the left of the panel. The shear in the panel will then be equal to the left reaction, $= R_1 = (5 \times 3 \times P)/7 = 15/7P$. The stress in 2–3 = $15/7P \cdot \sec\theta = +28.8$ tons, while the stress in 3–4 = $-15/7P \cdot \sec\theta = -28.8$ tons. The minimum stresses in 2–3 and 3–4 will occur when there is one load on the shorter segment. In the corresponding panel on the right of the truss, if the shorter segment is loaded, the left reaction = $1/7P$ = the shear in the panel. The minimum stress in 2–3 = $-1/7P \cdot \sec\theta = -1.92$ tons, while the minimum stress in 3–4 = $+1.92$ tons. The stresses in remaining panels are calculated in same manner.

(*c*) **Results.**—It will be seen that the web members meeting on the unloaded chord (top chord) have their maximum and minimum stresses for the same loading.

PROBLEM 9A. LIVE LOAD STRESSES IN A WARREN TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Warren truss, span 180′ 0″; panel length 20′ 0″, depth 24′ 0″, live load 1,500 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to the live loads by algebraic resolution. Scale of truss, 1″ = 25′ 0″.

PROBLEM 10. MAXIMUM AND MINIMUM STRESSES IN WARREN TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Warren truss, span 160′ 0″, panel length 20′ 0″, depth 20′ 0″, dead load 800 lb. per lineal foot per truss, live load 1,600 lb. per lineal foot per truss. Calculate the maximum and minimum stresses in the members due to dead and live loads by algebraic resolution.

(*b*) **Methods.**—Construct three truss diagrams as shown. On the first truss diagram place the dead load and the maximum live load chord coefficients, calculated as in Problems 8 and 9. The maximum live load chord coefficients are the same as the dead load chord coefficients. On the second diagram place the maximum and minimum live load web coefficients, calculated as in Problem 9. The maximum live load web coefficients are given on the left and the minimum live load coefficients are given on the right of the diagram. On the third diagram place the maximum and minimum stresses. The maximum chord stresses are equal to the sum of the dead and live load chord stresses. The minimum chord stresses are the dead load chord stresses. The maximum web stresses are the sum of the dead and maximum live load web stresses. The minimum web stresses are the algebraic sum of dead load stresses and minimum live load stresses.

(*c*) **Results.**—The web members 7–6 and 7–8 have a reversal of stress from tension to compression, or the reverse. These members must be counterbraced to take both kinds of stress.

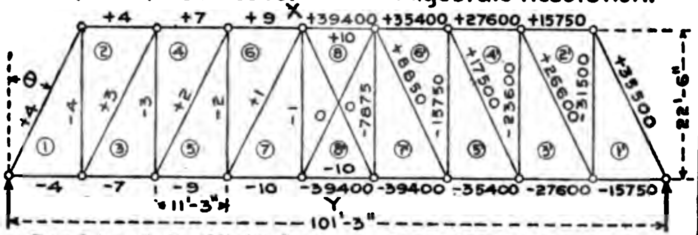
PROBLEM 10A. MAXIMUM AND MINIMUM STRESSES IN WARREN TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Warren truss, span 180′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 700 lb. per lineal foot per truss, live load 1,500 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution.

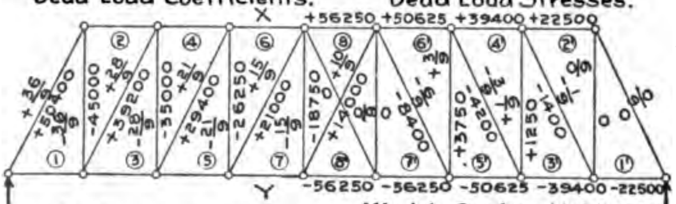
# Bridge Analysis. Problem 12.

## Howe Truss.

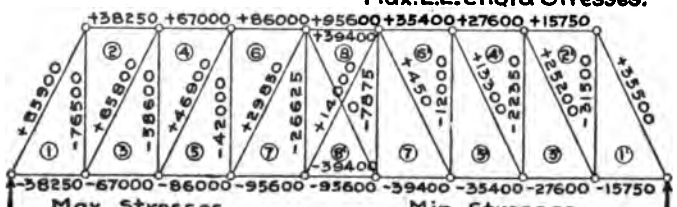
Max. and Min. Stresses. Algebraic Resolution.




Dead Load Coefficients. Dead Load Stresses.




Max. Coef. and Web Stresses. Min. L.L. Coef. and Web Str. Max. L.L. Chord Stresses.




Max. Stresses. Min. Stresses.

Dead Load = 700# per lineal ft. per truss. W = 7875.#

Live Load = 1000# " " " " " . P = 11250.#

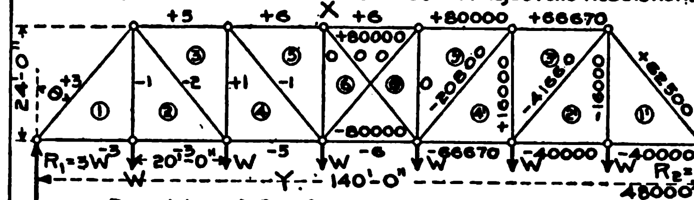
Span, L, = 101'-3". Panel, l, = 11'-3". Sec θ = 1.12.

Depth, d, = 22'-6". Tan θ = 0.50.

# Bridge Analysis. Problem 11.

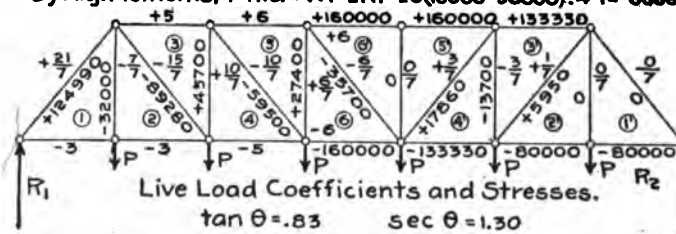
## Pratt Truss.

Maximum and Minimum Stresses. Algebraic Resolution.



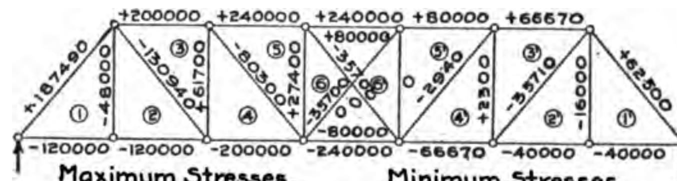

Dead Load Coef. Dead Load Stresses.

By Alg. Moments, 4-Yxd = Wl-2Rl = 20(16000-96000). 4-Y = -66667




Live Load Coefficients and Stresses.

tan θ = .83 sec θ = 1.30




Maximum Stresses. Minimum Stresses.

Dead Load, W = 16000# per joint. Live Load, P = 32000# per joint.

Span, L, = 140'-0". Panel, l, = 20'-0".

Depth, d, = 24'-0". No. of panels, N, = 7.

PROBLEM 11. MAXIMUM AND MINIMUM STRESSES IN A PRATT TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Pratt truss, span 140′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 800 lb. per lineal foot per truss, live load 1,600 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution.

(*b*) **Methods.**—Construct three truss diagrams as shown. On the first place the dead load coefficients and the dead load stresses. On the second place the live load coefficients and the live load stresses. On the third place the maximum and minimum stresses due to dead and live loads. The maximum chord stresses are the sums of the dead and live load chord stresses, while the minimum chord stresses are those due to dead load alone. The hip vertical is simply a hanger and has a minimum stress of one dead load and a maximum stress of one live and one dead load. Conditions for maximum and minimum stresses in webs are the same as for the Warren truss.

(*c*) **Results.**—There is no dead load shear in the middle panel, but it is seen that there are stresses in the counters for live loads. Only one of the counters will be in action at one time. Whenever the center of gravity of the loads is not in the center line of the truss, that counter will be acting that extends downward toward the center of gravity. The numerators of the maximum and minimum live load web coefficients are 0, 1, 3, 6, 10, 15, 21, as for the Warren truss. This shows that maximum and minimum web stresses are proportional to ordinates to a parabola.

PROBLEM 11A. MAXIMUM AND MINIMUM STRESSES IN A PRATT TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Pratt truss, span 160′ 0″, panel length 20′ 0″, depth 26′ 0″, dead load 700 lb. per lineal foot per truss, live load 1,500 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 25′ 0″.

PROBLEM 12. MAXIMUM AND MINIMUM STRESSES IN A HOWE TRUSS BY ALGEBRAIC RESOLUTION.

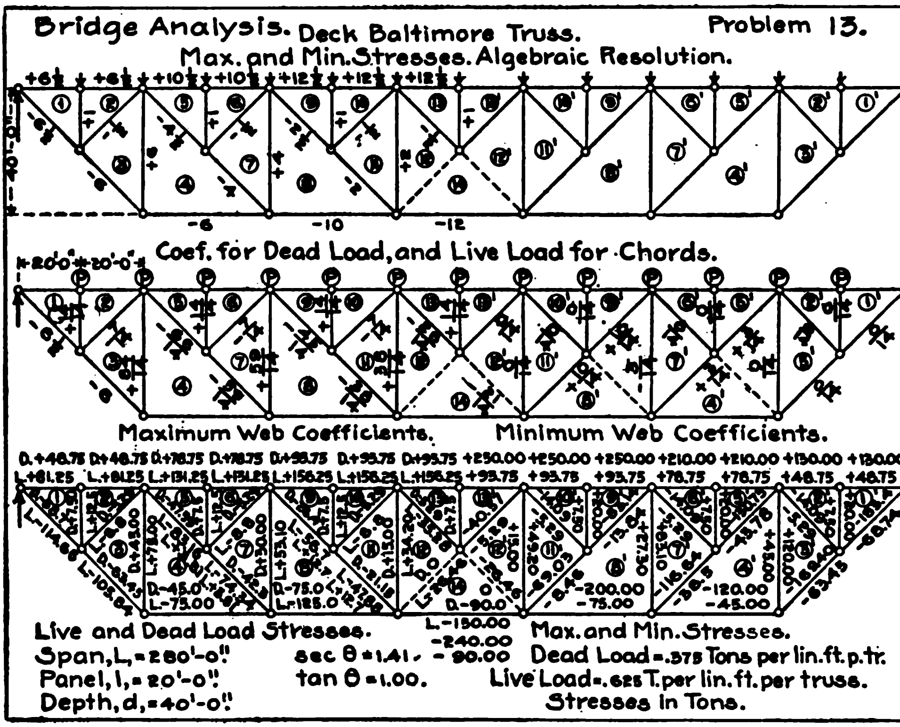
(*a*) **Problem.**—Given a Howe truss, span 101′ 3″, panel length 11′ 3″, depth 22′ 6″, dead load 700 lb. per lineal foot per truss, live load 1,000 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution.

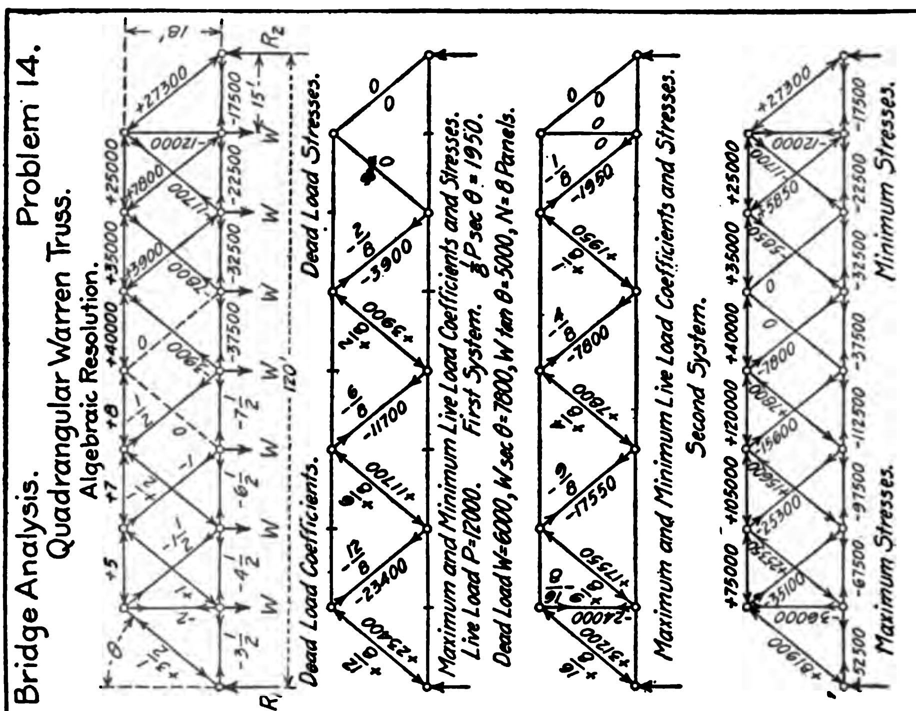
(*b*) **Methods.**—Construct three truss diagrams as shown. On the first diagram place the dead load coefficients and the dead load stresses. On the second diagram place the live load web coefficients and the maximum and minimum live load stresses. On the third diagram place the maximum and minimum stresses due to dead and live loads. The conditions for loading for the maximum and minimum stresses are the same as for a Pratt truss except that the vertical tie 1–2 carries the shear in the first panel and has a maximum stress for a full load on the truss.

(*c*) **Results.**—The vertical members are always in tension, while the diagonal members are always in compression. The web members meeting on the unloaded chord (top chord) have maximum and minimum stresses for the same loading. The counters in the center panel carry live load stress only, the counter acting downward away from the center of gravity of the loads being stressed. The maximum and minimum web stresses are the algebraic sums of the corresponding dead and live load stresses. The maximum chord stresses are the sums of the dead and live load chord stresses, while the minimum chord stresses are the dead load stresses alone.

PROBLEM 12A. MAXIMUM AND MINIMUM STRESSES IN A HOWE TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a Howe truss, span 120′ 0″, panel length 12′ 0″, depth 24′ 0″, dead load 700 lb. per lineal foot per truss, live load 1,000 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 20′ 0″.


Bridge Analysis. Deck Baltimore Truss. Problem 13.
Max. and Min. Stresses. Algebraic Resolution.
Coef. for Dead Load, and Live Load for Chords.
Maximum Web Coefficients. Minimum Web Coefficients.
Live and Dead Load Stresses.
Span, L, = 280'-0".
Panel, l, = 20'-0".
Depth, d, = 40'-0".
sec θ = 1.41.
tan θ = 1.00.
Max. and Min. Stresses.
Dead Load = .375 Tons per lin. ft. p. tr.
Live Load = .625 T. per lin. ft. per truss.
Stresses in Tons.


Bridge Analysis. Problem 14.
Quadrangular Warren Truss.
Algebraic Resolution.
Dead Load Coefficients.
Dead Load Stresses.
Maximum and Minimum Live Load Coefficients and Stresses.
First System.
Live Load P=12000.
$\frac{1}{8}$ P sec θ = 1950.
Dead Load W=6000, W sec θ=7800, W tan θ=5000, N=8 Panels.
Maximum and Minimum Live Load Coefficients and Stresses.
Second System.
Maximum Stresses.
Minimum Stresses.

### PROBLEM 13. MAXIMUM AND MINIMUM STRESSES IN A DECK BALTIMORE TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a deck Baltimore truss, span 280′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 0.375 tons per lineal foot per truss, live load 0.625 tons per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution.

(*b*) **Methods.**—Construct three truss diagrams and use them as shown.

*Dead Load Stresses.*—The auxiliary struts 1–2, 5–6, 9–10, etc., carry a full dead load compression, while the auxiliary web members 2–3, 6–7, 10–11, etc., have a tensile stress of $\frac{1}{2}W\cdot\sec\theta$. The stress in $1-Y$ equals the shear in the panel multiplied by $\sec\theta = -6\frac{1}{2}W\cdot\sec\theta$. The stress in $3-Y$ equals the shear in the panel multiplied by $\sec\theta$, plus the inclined component of the one-half load that is carried toward the center by the auxiliary member 2–3, $= -(5\frac{1}{2}+\frac{1}{2})W\cdot\sec\theta = -6W\cdot\sec\theta$. The stress in 3–4 is the vertical component of the stress in $3-Y = +6W$. The stress in $4-Y$ is the horizontal component of the stress in $3-Y = -6W\cdot\tan\theta$. The stress in $1-X$ and $2-X = +6\frac{1}{2}W\cdot\tan\theta$. The stress in 4–5 is the inclined component of the shear in the panel $= -4\frac{1}{2}W\cdot\sec\theta$. The stress in $5-X = -(-6-4\frac{1}{2})W\cdot\tan\theta = +10\frac{1}{2}W\cdot\tan\theta$. The remaining dead load stresses are calculated in a similar manner.

*Live Load Web Stresses.*—The maximum shears in the different panels occur when the longer segment of the truss is loaded, while the minimum shears occur when the shorter segment of the truss is loaded. The maximum stresses in the webs in the first and second panels occur for a full live load on the bridge. The maximum shear in the third panel occurs with all loads to the right of the panel and no loads to the left. The shear in the panel will then be equal to the left reaction $= 11\times\frac{1}{2}(11+1)P/14 = 66/14P$. The maximum live load stress in 4–5 will be $= -66/14\,P\cdot\sec\theta$. With a maximum stress in 4–5 the stress in 4–7 will be $= (-66/14+7/14)P\cdot\sec\theta = -59/14P\cdot\sec\theta$. This is the maximum stress, for the stress in 4–7 when there is a maximum shear in the panel is $= 10\times 11/2\times 1/14P\cdot\sec\theta = -55/14P\cdot\sec\theta$. In a similar manner it will be found that maximum stresses in members 8–9 and 8–11 occur with a maximum shear in 8–9. On the right side it will be seen that minimum stresses in the diagonals occur for a minimum shear in the odd-numbered panels from the right.

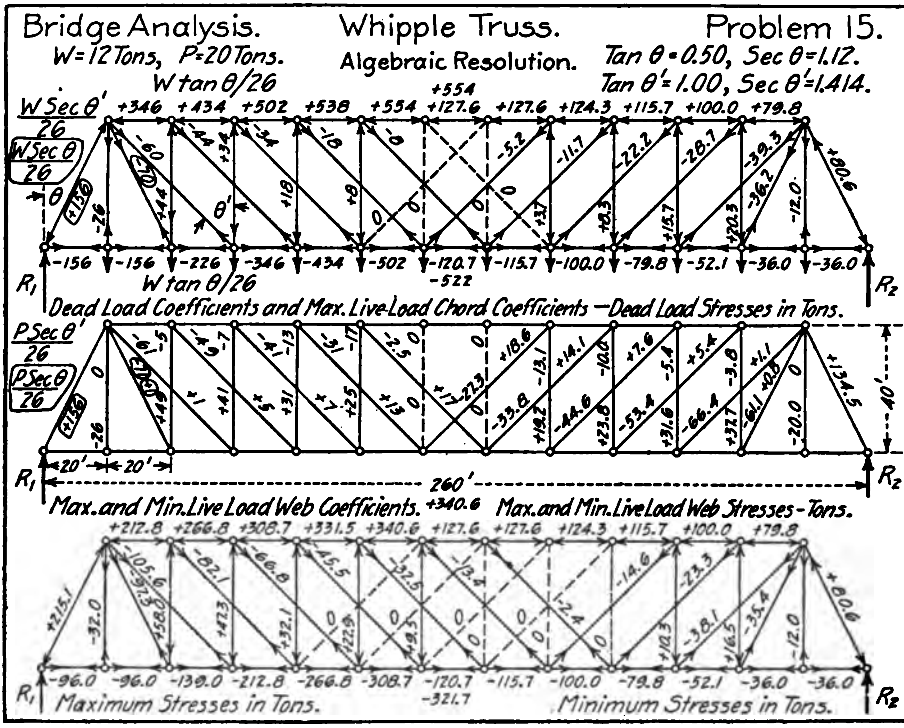
(*c*) **Results.**—The dead and live loads were assumed as applied on the upper chord. The upper chords are in compression, while the lower chords are in tension the same as for a through truss. The live and dead load stresses are given separately on the left side of the lower truss as required by Cooper's Specifications.

### PROBLEM 13A. MAXIMUM AND MINIMUM STRESSES IN A DECK BALTIMORE TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a deck Baltimore truss, span 320′ 0″, panel length 20′ 0″, depth 50′ 0″, dead load 0.3 tons per lineal foot per truss, live load 0.5 tons per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 40′ 0″.

### PROBLEM 14. MAXIMUM AND MINIMUM STRESSES IN A QUADRANGULAR WARREN TRUSS BY ALGEBRAIC RESOLUTION.

(*a*) **Problem.**—Given a quadrangular Warren truss, span 120′ 0″, panel length 15′ 0″, depth 18′ 0″, dead load 400 lb. per lineal foot per truss, live load 800 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 20′ 0″.




(*b*) **Methods.**—Construct four truss diagrams as shown.

*Dead Load Stresses.*—The left reaction is $3\frac{1}{2}W$ and the stresses in the end-post and end panel in the bottom chord are $+3\frac{1}{2}W\cdot\sec\theta$ and $-3\frac{1}{2}W\cdot\tan\theta$, respectively. The other stresses are indeterminate in working from the abutment, and it is necessary to pass to the center of the truss. The two diagonals meeting at the middle of the top chord will have no stress for dead load, and the load at the middle of the truss will be equally divided between the two diagonals meeting at the center of the bottom chord. Passing to the left, the first load to the left of the center is carried directly to the left abutment as shown, while the corresponding load on the right of the center is carried directly to the right abutment. The remaining shears can now be calculated. The chord stresses can now be calculated as in the case of a single intersection truss.

*Live Load Stresses.*—The coefficients of the maximum chord stresses are the same as the dead load chord coefficients. The maximum and minimum live load web stresses are not statically determinate, but the following solution gives approximate results: Divide the truss into two systems, the first carrying three full loads and the second carrying four full loads as shown. Then calculate the maximum and minimum web stresses in the two systems separately in the usual manner. The maximum web stresses will occur with the longer segment loaded, while the minimum web stresses will occur with the shorter segment loaded. The maximum and minimum stresses are given in the fourth diagram.

(*c*) **Results.**—The live load web stresses as calculated above are approximate, but are on the safe side. The exact solution can be made only by an application of the "Theory of Least Work." This type of truss, with riveted connections, is used by the American Bridge Company for spans of 80 to 170 feet.

### Problem 14A. Maximum and Minimum Stresses in a Quadrangular Warren Truss by Algebraic Resolution.

(*a*) **Problem.**—Given a quadrangular Warren truss, span 135′ 0″, panel length 15′ 0″, depth 20′ 0″, dead load 400 lb. per lineal foot per truss, live load 700 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 20′ 0″.

### Problem 15. Maximum and Minimum Stresses in a Whipple Truss by Algebraic Resolution.

(*a*) **Problem.**—Given a Whipple truss, span 260′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 1,200 lb. per lineal foot per truss, live load 2,000 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 30′ 0″.

(*b*) **Methods.**—The dead load stresses and the maximum live load chord stresses can be calculated by beginning at the center and calculating the shears, and then calculating the chord stresses as in Problem 14. The maximum and minimum live load web stresses are statically indeterminate as were the web stresses in Problem 14. The usual solution of this problem is to divide the truss into two trusses of single intersection. The dead and the live load chord stresses and the maximum and minimum web stresses are then calculated as for independent trusses. The loads at the foot of the hip verticals are assumed as equally divided between the two systems. The final chord stresses are the sums of the chord stresses in the separate trusses. The stresses in the web members, except the hip vertical, are as given in the separate trusses. In solving the problem the partial truss diagrams should be drawn. The trusses will be unsymmetrical, one being the same as the other turned end for end. With the joints all loaded the dead load chord and web coefficients, and the live load chord coefficients are calculated. In calculating the maximum live load web coefficients the loads are moved off to the right, and the maximum stresses in the webs on the left of the center will occur when the longer segment is loaded, and the minimum stresses in the webs on the right will occur when the shorter segment is loaded. Then with all joints in the truss loaded move the loads off to the left, calculating the maximum web coefficients on the right of the center and the minimum web coefficients on the left of the center. In calculating the stresses from the shears it will be seen that functions of two angles are used. The relation between the two angles is $\tan \theta' = 2 \tan \theta$. Web coefficients in terms of $\theta$ are enclosed in a ring. The calculation of the chord coefficients may be illustrated by calculating the coefficient of the end panel of the upper chord $= -[-156/26 - 70/26 - 2(60/26)]W \cdot \tan \theta = 346/26 W \cdot \tan \theta$.

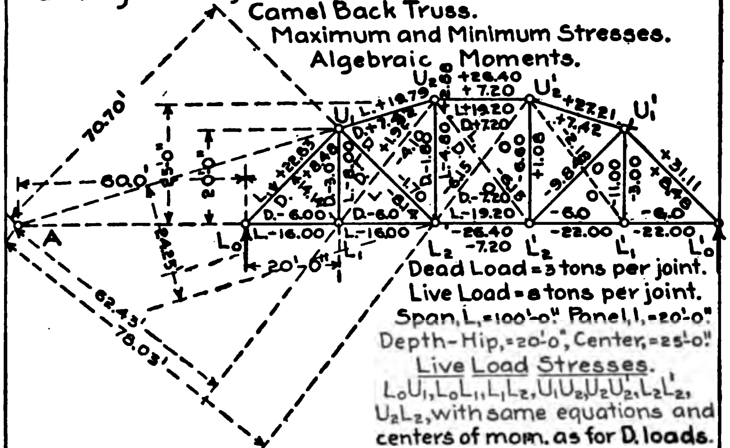
(*c*) **Results.**—The chord stresses calculated as above do not agree with those calculated by beginning at the center of the truss as in Problem 14. The student should calculate the dead load chord and web stresses and the live load chord stresses as in Problem 14. Whipple trusses were usually built with an odd number of panels. The Whipple truss was formerly quite generally used for long span highway and railway bridges, but is now rarely built, being replaced by the Petit truss.

### Problem 15A. Maximum and Minimum Stresses in a Whipple Truss by Algebraic Resolution.

(*a*) **Problem.**—Given a Whipple truss, span 260′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 1,200 lb. per lineal foot per truss, live load 2,000 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads. Calculate the dead load chord and web stresses and the live load chord stresses as in Problem 14. Scale of truss, 1″ = 30′ 0″.

## Bridge Analysis. Problem 17.

### Camel Back Truss.
### Maximum and Minimum Stresses.
### Algebraic Moments.




Dead Load = 3 tons per joint.
Live Load = 8 tons per joint.
Span, $L_1$ = 100'-0". Panel, $l_1$ = 20'-0".
Depth - Hip, = 20'-0", Center, = 25'-0".

**Dead Load Stresses.**

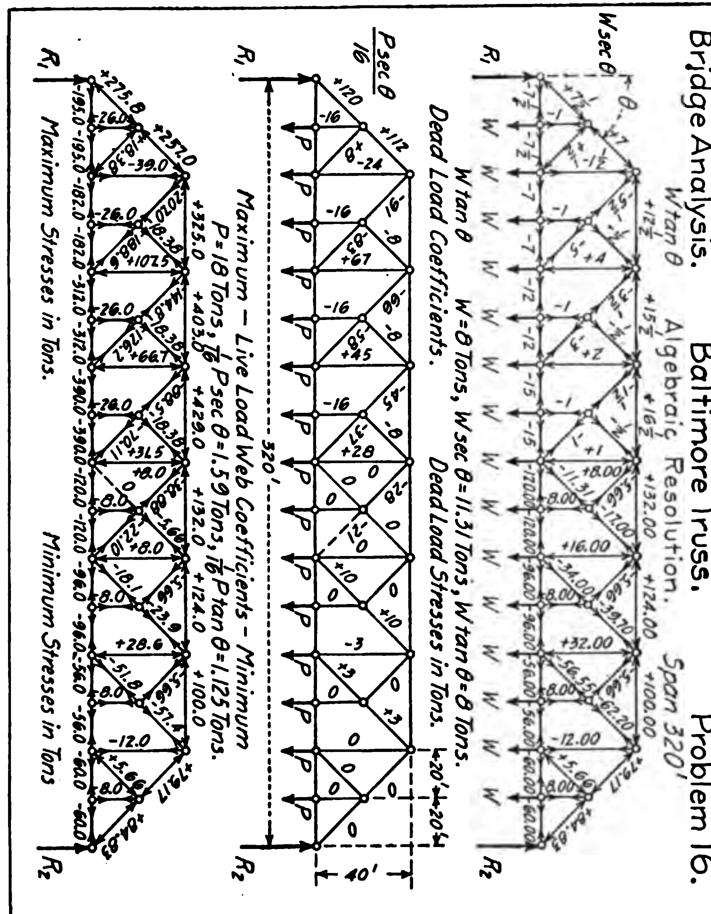
$L_0U_1$ - Moments about $L_1$.
$L_0U_1 \times 14.14 = R_1 \times 20.0 = 0$. $L_0U_1 = +8.48$.
$L_0L_1$ and $L_1L_2$. Moments about $U_1$.
$-L_0L_1 \times 20.0 - R_1 \times 20.0 = 0$. $L_0L_1 = -6.0$.
$U_1U_2$. - Moments about $L_2$.
$U_1U_2 \times 24.25 - R_1 \times 40 + W \times 20 = 0$. $U_1U_2 = +7.42$.
$U_2U'_2$. - Moments about $L_2$.
$U_2U'_2 \times 25.0 - R_1 \times 40 + W \times 20 = 0$. $U_2U'_2 = +7.2$.
$U_1L_2, U_2L_1, U_2L_2$. - Moments at A.
$U_1L_2 \times 70.70 + R_1 \times 60 - W \times 80 = 0$. $U_1L_2 = -1.70$.
$-U_2L_1 \times 62.43 + R_1 \times 60.0 - W \times 80.0 = 0$.
$U_2L_1 = +1.92$.
$-U_2L_2 \times 100 + R_1 \times 60 - W \times 80 - W \times 100 = 0$. $U_2L_2 = -1.80$.

**Live Load Stresses.**

$-U'_2L_2 \times 78.03 + 11.2 \times 60 - 8 \times 180 + 288 = 0$.
$U'_2L_2 = -6.15$.

**Live Load Stresses.**

$L_0U_1, L_0L_1, L_1L_2, U_1U_2, U_2U'_2, L_2L'_2$, $U_2L_2$, with same equations and centers of mom. as for D. loads.
$L_0U_1 = +22.63$; $L_0L_1 = L_1L_2 = -16.0$;
$U_1U_2 = +19.79$; $U_2U'_2 = +19.20 = -L_2L'_2$;
$U_2L_2 = -4.80$. $U_1L_2$, Loads $L_2, L'_2$, $L'_1$ for Maximum.
$U_1L_2 \times 70.70 + R_1 \times 60 = 0$
$U_1L_2 \times 70.7 + 9.6 \times 60 = 0$. $U_1L_2 = -8.14$.
$U_1L_2$, Load $L_1$ for Minimum.
$U_1L_2 \times 70.7 + R_1 \times 60 - P \times 80 = 0$. $+3.62$
$U_1L_2 \times 70.7 + 6.4 \times 60 - 8 \times 80 = 0$. $U_1L_2 = ?$
Since +3.62 is greater than -1.70 counter $U_2L_1$ will act and, -
$-U_2L_1 \times 62.43 + 6.4 \times 60 - 8 \times 80 = 0$. $U_2L_1 = -4.10$. $U_2L_2$, Loads $L'_2$ and $L'_1$ for Min. $-U_2L_2 \times 100 + R_1 \times 60 = 0$. $U_2L_2 = +2.88$. $U'_2L_2$, Loads $L_1$ and $L_2$ for Max. $-U'_2L_2 \times 78.03 + R_1 \times 60 - P(80+100) + U_2U'_2 \times 25 = 0$. $U_2U'_2$ (Mom. $L_2$) = +11.5.



### Problem 16. Maximum and Minimum Stresses in a Through Baltimore Truss by Algebraic Resolution.

(*a*) **Problem.**—Given a through Baltimore truss, span 320′ 0″, panel length 20′ 0″, depth 40′ 0″, dead load 800 lb. per lineal foot per truss, live load 1,800 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 40′ 0″.

(*b*) **Methods.**—Construct three truss diagrams as shown.

*Dead Load Stresses.*—The shear in each of the hangers is $W$, while the stress in each of the diagonal auxiliary members is $-\frac{1}{2}W\cdot\sec\theta$. The stress in the upper part of the end-post is $(+6\frac{1}{2}+\frac{1}{2})W\cdot\sec\theta = +7W\cdot\sec\theta$, where $+6\frac{1}{2}W\cdot\sec\theta$ is the stress due to the shear and $+\frac{1}{2}W\cdot\sec\theta$ is the stress due to the half load carried toward the center by the auxiliary diagonal member. The stress in the main diagonal in the third panel is $-5\frac{1}{2}W\cdot\sec\theta$, where $5\frac{1}{2}W$ is the shear in the panel; while the stress in the diagonal in the fourth panel is $(-4\frac{1}{2}-\frac{1}{2})W\cdot\sec\theta = -5W\cdot\sec\theta$, where $4\frac{1}{2}W\cdot\sec\theta$ is the stress due to the shear in the panel and $\frac{1}{2}W\cdot\sec\theta$ is the stress carried toward the center of the truss by the auxiliary member. The chord coefficients are calculated as in Problem 13.

*Live Load Stresses.*—The maximum shear in the third panel occurs with 13 loads to the right of the panel and with no loads to the left of the panel. The shear in the panel is then equal to the left reaction, equals $13 \times \frac{1}{2}(13+1) \times P/16 = \frac{91}{16}P$. The stress in the main diagonal in the third panel is then equal to $-\frac{91}{16}P\cdot\sec\theta$. The stress in the main diagonal in the fourth panel is $(-\frac{78}{16}P + \frac{8}{16}P)\sec\theta = -\frac{86}{16}P\cdot\sec\theta$, = a maximum, the maximum shear in the panel being $12 \times \frac{1}{2}(12+1) \times P/16 = \frac{78}{16}P$. In like manner the maximum stresses are found in the 5th and 6th panels when there is a maximum shear in the 5th panel, and in the 7th and 8th panels when there is a maximum shear in the 7th panel. Minimum stresses in the 3d and 4th panels from the right abutment occur when there is a minimum shear in the 3d panel; and in the 5th and 6th panels when there is a minimum shear in the 5th panel.

(*c*) **Results.**—The double panels next to the center require counters. It should be noticed that in calculating the stresses in these counters the diagonal auxiliary ties will have the dead load stress of + 5.66 tons as a minimum.

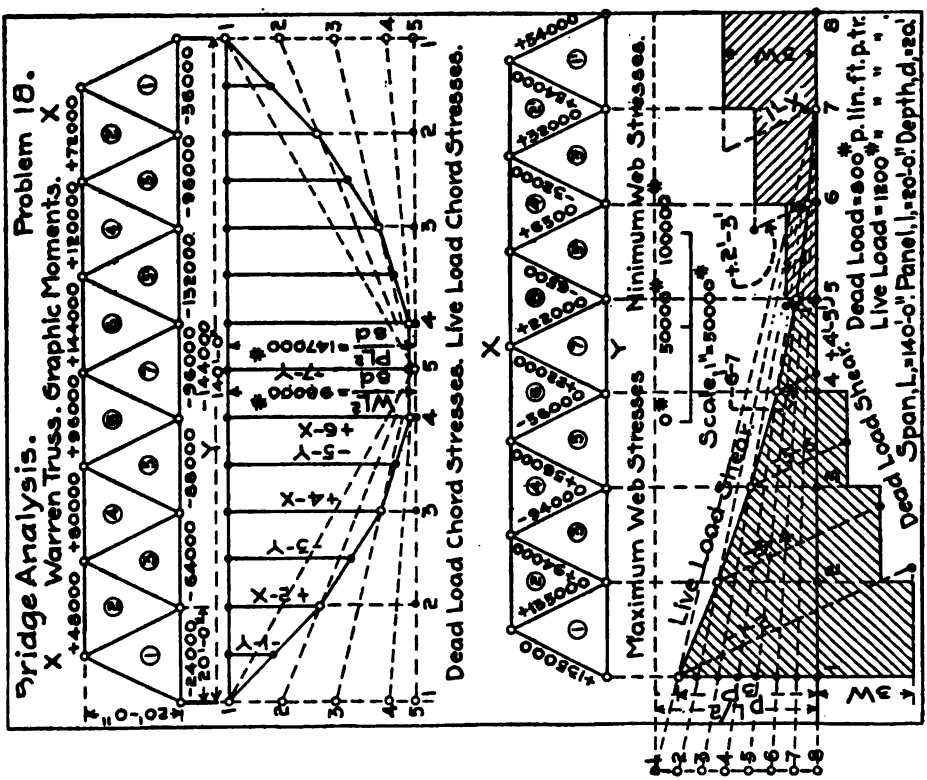
### Problem 16a. Maximum and Minimum Stresses in a Through Baltimore Truss by Algebraic Resolution.

(*a*) **Problem.**—Given a through Baltimore truss, span 320′ 0″, panel length 20′ 0″, depth 45′ 0″, dead load 800 lb. per lineal foot per truss, live load 1,800 lb. per lineal foot per truss. All the auxiliary ties are to be in compression as in the 1st and 2d panels in Problem 16 and as in Problem 6. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss, 1″ = 40′ 0″.

### Problem 17. Maximum and Minimum Stresses in a Camel-back Truss by Algebraic Moments.

(*a*) **Problem.**—Given a Camel-back truss, span 100′ 0″, panel length 20′ 0″, depth at hip 20′ 0″, depth at center 25′ 0″, dead load 300 lb. per lineal foot per truss, live load 800 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic moments. Scale of truss, 1″ = 20′ 0″.

(*b*) *Methods.*—Calculate the arms of the forces as shown and check the values by scaling from the drawing.




*Dead Load Stresses.*—To calculate the stress in the end-post $L_0U_1$, take center of moments at $L_1$, and pass a section cutting $L_0U_1$, $U_1L_1$ and $L_1L_2$, and cutting away the truss to the right. Then assume stress $L_0U_1$ as an external force acting from the outside toward the cut section, and stress $L_0U_1 \times 14.14 - R_1 \times 20 = 0$. Now $R_1 = 6$ tons and stress $L_0U_1 = +8.48$ tons. To calculate the stresses in $L_0L_1$ and $L_1L_2$ take the center of moments at $U_1$, and pass a section cutting members $U_1U_2$, $U_1L_2$ and $L_1L_2$, and cutting away the truss to the right. Then assume the stress in $L_1L_2$ as an external force acting from the outside toward the cut section, and $L_1L_2 \times 20 - R_1 \times 20 = 0$. Now $R_1 = 6$ tons and the stress in $L_0L_1 = L_1L_2 = -6$ tons. To calculate the stress in $U_1U_2$ take the center of moments at $L_2$, and pass a section cutting members $U_1U_2$, $U_2L_2$ and $L_2L_2'$, and cutting away the truss to the right. Then assume the stress in $L_1U_2$ as an external force acting from the outside toward the cut section, and $U_1U_2 \times 24.25 - R_1 \times 40 + W \times 20 = 0$. Now $R_1 = 6$, $W = 3$ tons, and the stress in $U_1U_2 = +7.42$ tons. To calculate the stress in $U_1L_2$ take the center of moments at $A$, and pass a section cutting members $U_1U_2$, $U_1L_2$, and $L_1L_2$, and cutting away the truss to the right. Then assume the stress in $U_1L_2$ as an external force acting from the outside toward the cut section, and $U_1L_2 \times 70.7 + R_1 \times 60 - W \times 80 = 0$. Now $R_1 = 6$ tons and $W = 3$ tons, and $U_1L_2 \times 70.7 = -120$ ft.-tons, and stress $U_1L_2 = -1.70$ tons. The other dead load stresses are calculated as shown.

*Live Load Stresses.*—The live load chord stresses are equal to the dead load chord stresses multiplied by 8/3. The maximum stress in $U_1L_2$ will occur with loads at $L_2$, $L_2'$, and $L_1'$, while the maximum stress in counter $U_2L_1$ will occur with a load at $L_1$ only. The maximum tension in $U_2L_2$ will occur with all the live loads on the bridge, while the maximum compression will occur when there is a maximum stress in the counter $U_2L_2'$, loads at $L_2'$ and $L_1'$. The details of the solution are shown in the problem.

(c) **Results.**—The stress in the counter $U_2L_2'$ and the chords $U_2U_2'$ and $L_2L_3'$ may be calculated by the method of coefficients, and will be the same as for a truss with parallel chords having a depth of 25′ 0″. The maximum stress in $U_2L_2'$ will occur with loads $L_2'$ and $L_1'$ on the bridge, when the left reaction equals $2 \times 3P/5 = 6/5P$. The stress in $U_2L_2' = -6/5P \cdot \sec\theta = -6.15$ tons.

### Problem 17a. Maximum and Minimum Stresses in a Camel-back Truss by Algebraic Moments.

(a) **Problem.**—Given a Camel-back truss, span 120′ 0″, panel length 20′ 0″, depth at hip 25′ 0″, depth at $U_2$ 30′ 0″, depth at $U_3$ 30′ 0″, dead load 300 lb. per lineal foot per truss, live load 800 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads.

### Problem 18. Maximum and Minimum Stresses in a Through Warren Truss by Graphic Moments.

(a) **Problem.**—Given a through Warren truss, span 140′ 0″, panel length 20′ 0″, depth 20′ 0″, dead load 800 lb. per lineal foot per truss, live load 1,200 lb. per lineal foot per truss. Calculate the maximum and minimum stresses by graphic moments. Scale of truss, 1″ = 20′ 0″. Scale of loads, 1″ = 50,000 lb.

(b) **Methods.** *Chord Stresses.*—Calculate the center ordinate of the parabola $= w \cdot L^2/8d$ = 98,000 lb., and lay it off at 5 to the prescribed scale. Now lay off the vertical line 1–5 at the left and right abutments. Make 1–2 = 2–3 = 3–4 = 2 (4–5). Draw the inclined lines 1–5, 2–5, 3–5, 4–5, 5–5. The intersections of these lines with verticals let drop from the lower chord points are points in the stress parabola for the upper chord stresses. The stresses in the lower chords are the arithmetical means of the stresses in the upper chords on each side. By changing the scale the live load stresses may be scaled directly from the diagram.

*Web Stresses.*—At the distance of a panel to the left of the left abutment lay off the vertical line 1–8 equal to one-half the total live load on the truss, to the prescribed scale, equal 1,200 × 70 = 84,000 lb. Now divide the line 1–8 into as many equal parts as there are panels in the truss, and mark the points of division 2, 3, 4, etc. Connect these points of division with the panel point 7, the first panel point to the left of the right abutment. Drop verticals from the panel points of the lower chord of the truss to the line 1–8, and the intersections of like numbered lines will give points on the curve of maximum live load shears.

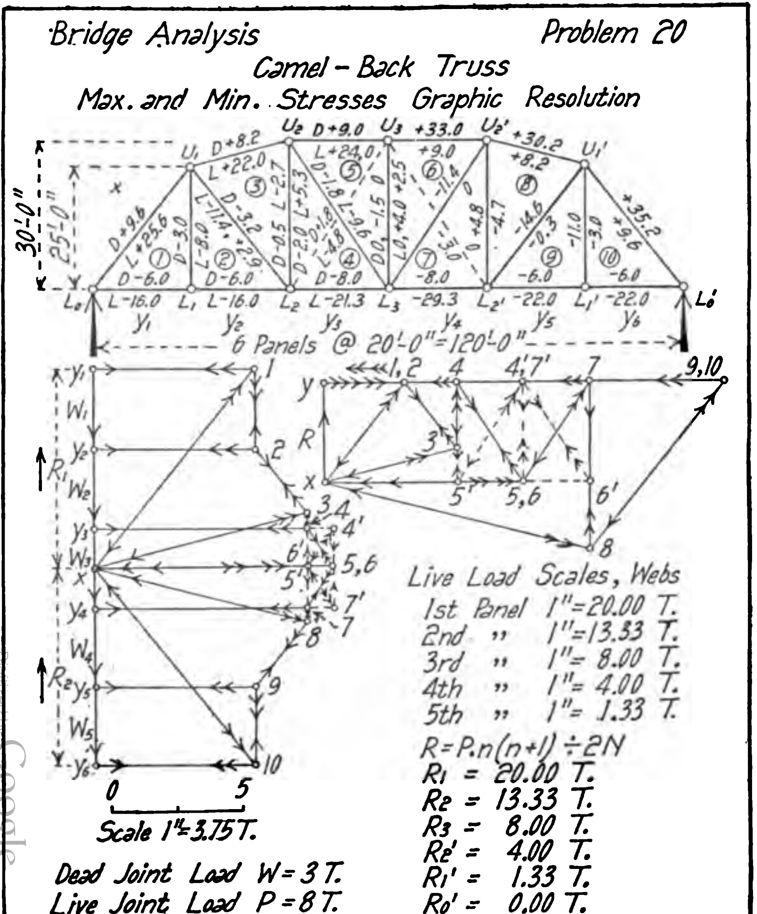
To construct the dead load shear diagram, lay off $3W$, downward to the prescribed scale under the left abutment, and reduce the shear under each load to the right by $W$, until the dead load shear is $-3W$ at the right abutment. The dead load shear diagram is then constructed as shown.

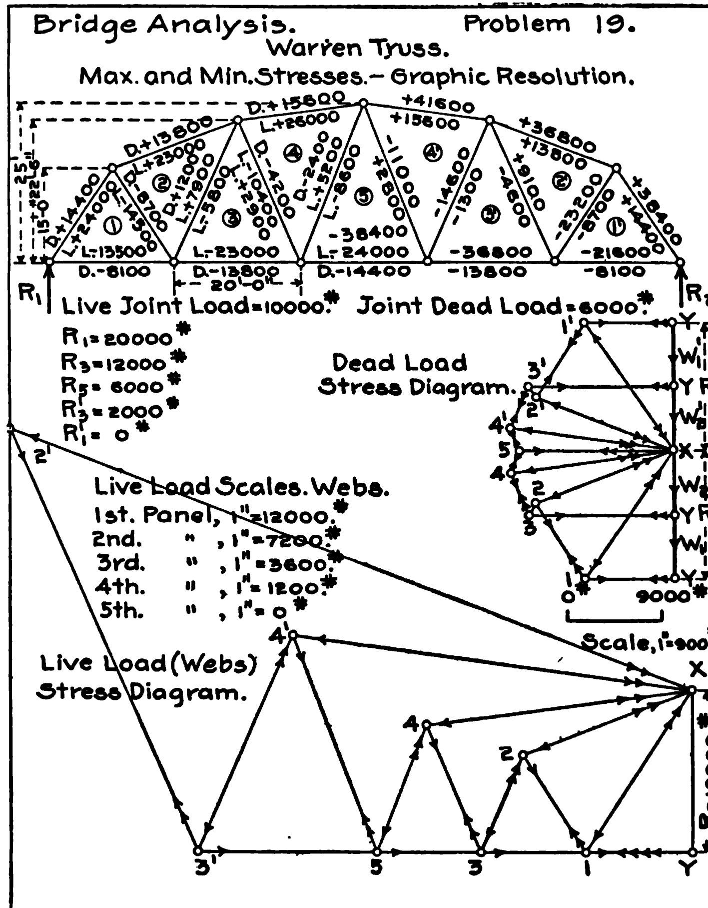
*Maximum and Minimum Web Stresses.*—The maximum shear in any panel is then the ordinate to the right of the panel point on the left end of the panel, and the stresses in the web members are calculated by drawing lines parallel to the corresponding member as shown. Positive stresses are measured downwards from the live load shear curve, and negative stresses are measured upwards from the live load shear curve.

(c) **Results.**—This method is an excellent one for illustrating the effect of the different systems of loads, but consumes too much time to be of practical use. It should be noted that the maximum ordinate to the chord parabola is not a chord stress in a Warren truss with an odd number of panels.

### Problem 18a. Maximum and Minimum Stresses in a Through Warren Truss by Graphic Moments.

(a) **Problem.**—Given a through Warren truss, span 160′ 0″, panel length 20′ 0″, depth 24′ 0″, dead load 900 lb. per lineal foot per truss, live load 1,200 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by graphic moments. Scale of truss, 1″ = 25′ 0″. Scale of loads, 1″ = 50,000 lb.


Bridge Analysis
Problem 20
Camel-Back Truss
Max. and Min. Stresses Graphic Resolution
6 Panels @ 20'-0" = 120'-0"
Live Load Scales, Webs
1st Panel 1" = 20.00 T.
2nd " 1" = 13.33 T.
3rd " 1" = 8.00 T.
4th " 1" = 4.00 T.
5th " 1" = 1.33 T.
R = P.n(n+1) ÷ 2N
R1 = 20.00 T.
R2 = 13.33 T.
R3 = 8.00 T.
R2' = 4.00 T.
R1' = 1.33 T.
R0' = 0.00 T.
Scale 1" = 3.75 T.
Dead Joint Load W = 3 T.
Live Joint Load P = 8 T.


Bridge Analysis.
Problem 19.
Warren Truss.
Max. and Min. Stresses.– Graphic Resolution.
Live Joint Load = 10000.#
Joint Dead Load = 6000.#
R1 = 20000#
R3 = 12000#
R5 = 6000#
R3' = 2000#
R1' = 0#
Dead Load
Stress Diagram.
Live Load Scales. Webs.
1st. Panel, 1" = 12000.#
2nd. " , 1" = 7200.#
3rd. " , 1" = 3600.#
4th. " , 1" = 1200.#
5th. " , 1" = 0#
Scale, 1" = 9000#
Live Load (Webs)
Stress Diagram.

PROBLEM 19. MAXIMUM AND MINIMUM STRESSES IN AN INCLINED CHORD THROUGH WARREN TRUSS BY GRAPHIC RESOLUTION.

(a) **Problem.**—Given an inclined chord through Warren truss, span 100′ 0″, panel length 20′ 0″, depth at the hip 15′ 0″, depth at the second panel 22′ 6″, depth at the center 25′ 0″, dead load 600 lb. per lineal foot per truss, live load 1,000 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by graphic resolution. Scale of truss, 1″ = 15′ 0″. Scale of dead loads, 1″ = 9,000 lb. Scale of live loads as shown.

(b) **Methods.**—Construct a truss diagram and calculate the dead load stresses in the usual way as shown. The live load chord stresses are found by multiplying the dead load chord stresses by 5/3. To calculate the maximum and minimum web stresses proceed as follows: Assume that the truss is fixed at the right abutment and that the left reaction is $R_1$ = say 10,000 lb. with no loads on the bridge. Then beginning at the left reaction $R_1$, calculate by graphic resolution the stresses in the different members of the truss due to the left reaction of 10,000 lb., there being no loads on the bridge. The reaction is laid off to a scale of 1″ = 6,000 lb. Now to calculate the maximum live load stress in any web member multiply the stress as scaled from the diagram by the ratio of the left reaction which produces the maximum stress to 10,000 lb. For example, the member 1–2 has a maximum stress with all the joints loaded and the reaction is 20,000 lb., or the scale of the stress is 1″ = 12,000 lb. The stress 1–2 then equals − 14,500 lb. The maximum live load stress in 2–3 occurs with loads at the three panel points at the right, and $R_1 = \frac{1}{5}(3 \times 2P)$ = 12,000 lb., or the scale of the stress in the diagram is 1″ = 7,200 lb., and the stress in 2–3 equals + 7,900 lb. The stresses in the remaining web members are calculated in the same manner.

(c) **Results.**—This solution may be used to calculate the maximum and minimum stresses in any truss, but it is best adapted to the solution of stresses in trusses like the one shown. The maximum and minimum stresses are given on the right hand side of the truss diagram.

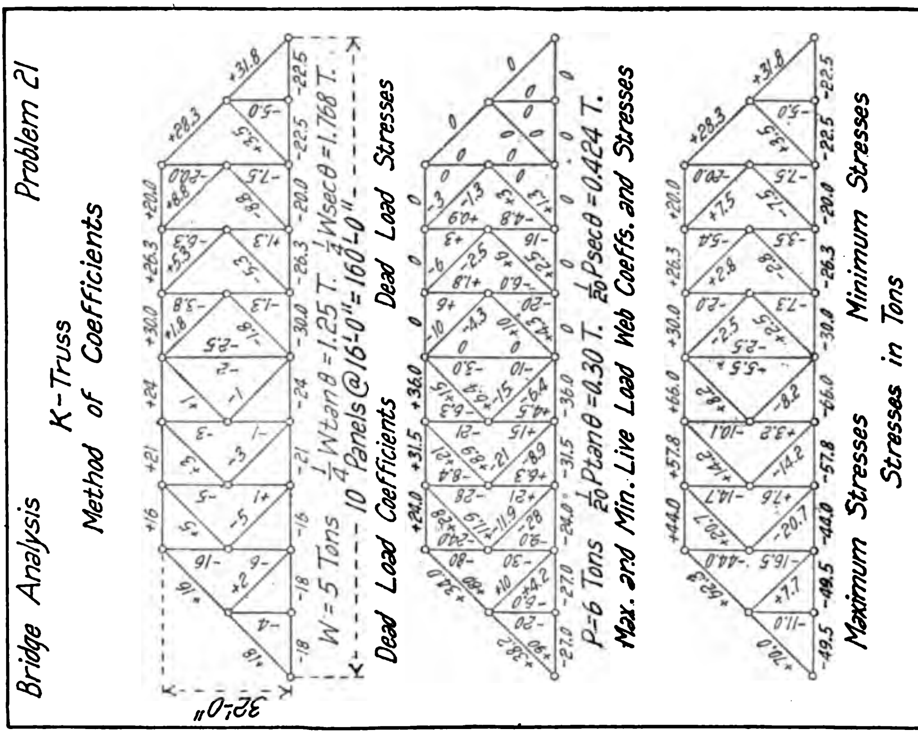
PROBLEM 19A. MAXIMUM AND MINIMUM STRESSES IN AN INCLINED CHORD THROUGH WARREN TRUSS BY GRAPHIC RESOLUTION.

(a) **Problem.**—Given an inclined chord through Warren truss, span 120′ 0″, panel length 20′ 0″, depth at the hip 15′ 0″, depth at the second panel in the top chord 22′ 6″, depth at the third panel 25′ 0″ (middle panel has parallel chords), dead load 600 lb. per lineal foot per truss, live load 1,100 lb. per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by graphic resolution. Scale of truss, 1″ = 20′ 0″. Scale of dead loads, 1″ = 10,000 lb. Scale of live loads as calculated.

PROBLEM 20. MAXIMUM AND MINIMUM STRESSES IN AN INCLINED CHORD THROUGH PRATT TRUSS BY GRAPHIC RESOLUTION.

(a) **Problem.**—Given an inclined chord through Pratt truss, span 120′ 0″, panel length 20′ 0″, depth at hip 25′ 0″, depth at second panel point 30′ 0″, dead joint load 3 tons per truss, live joint load 8 tons per truss. Calculate the maximum and minimum stresses due to dead and live loads by graphic resolution. Scale of truss, 1″ = 20′ 0″. Scale of dead loads, 1″ = 3.75 tons. Scale of live loads as calculated.

(b) **Methods.**—Construct a truss diagram and calculate the dead load stresses in the usual way as shown. The live load chord stresses are calculated by multiplying the dead load stresses by 8/3. To calculate the maximum and minimum web stresses proceed as follows: Assume the truss as fixed at the right abutment and that the left reaction = say 20 tons with no loads on the bridge. Then, beginning at the left reaction $R_1$, calculate by graphic resolution the stresses in the different members of the bridge due to the left reaction of 20 tons, there being no loads on the bridge. The reaction is laid off to a scale of 1″ = 20 tons. Now to calculate the maximum stress in any web member multiply the stress as scaled from the diagram by the ratio of the left reaction which produces the maximum stress to 20 tons. For example, the member 1–x has a



maximum stress with all joints loaded and the reaction is 20 tons and the scale of the stress is 1″ = 20 tons. The stress in 2–3 is scaled from the diagram and is + 25.6 tons. The maximum live load stress in 2–3 occurs with loads on four panel points to the right, and $R_2 = (2\frac{1}{2} \times 4P)/6$ = 13.33 tons, or the scale of the stress in the diagram is found by the proportion x : 13.33 :: 20: 20, and $x$ = 13.33 tons per inch. The stress in 2–3 = − 11.4 tons. The live load stresses in the remaining web members are calculated in the same manner. When the counters are acting there will be no dead load stresses in the main tie in the same panel. This makes it necessary to calculate the dead load stresses in the counters and in the posts when the counters are acting. The calculations for the dead load stresses in the counters and in the posts when the counters are acting have been calculated in the dead load stress diagram and are shown by the primes. The maximum and minimum stresses are found by combining the dead and live load stresses in the same manner as when the stresses are calculated by algebraic moments.

(a) **Results.**—This solution is an excellent one to use in checking the results obtained by algebraic methods. The maximum and minimum stresses are given on the right hand side of the truss diagram.

### PROBLEM 20A. MAXIMUM AND MINIMUM STRESSES IN AN INCLINED CHORD THROUGH PRATT BY GRAPHIC RESOLUTION.

(a) **Problem.**—Given an inclined chord through Pratt truss, span 100′ 0″, panel length 20′ 0″, depth at hip 25′ 0″, depth at second panel 30′ 0″, dead joint load 3 tons per truss, live joint load 8 tons per truss. Calculate the maximum and minimum stresses due to dead and live loads by graphic resolution. Scale of truss 1″ = 20′ 0″. Scale of dead loads 1″ = 3 tons. Scale of live loads as calculated.

PROBLEM 21. MAXIMUM AND MINIMUM STRESSES IN A THROUGH K-TRUSS BY ALGEBRAIC RESOLUTION (METHOD OF COEFFICIENTS).

(a) **Problem.**—Given a through K-truss, span 160′ 0″, panel length 16′ 0″, depth 32′ 0″, dead load 5 tons per joint per truss, live load 6 tons per joint per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss 1″ = 24′ 0″.

(b) **Methods.**—Construct three truss diagrams as shown.

*Dead Load Stresses.*—The horizontal components of the stress in the diagonals in the third and following panels must be equal but opposite in direction. The shear in the panel is therefore equally divided between the two diagonals. The coefficients are given in fourths of $W$ for convenience. The stress in the first hanger will be $-4$ fourths, and the coefficient of the stress in the diagonal will be $+2$ fourths. The shear in the third panel will be 18 fourths minus 8 fourths, which is 10 fourths. The coefficient of the stress in the upper and lower diagonals will be $+5$ and $-5$ fourths, respectively. In like manner, the coefficients of the stress in the upper and lower diagonals in the fourth panel will be $+3$ fourths and $-3$ fourths, respectively; and the coefficients in the fifth panel $+1$ fourth and $-1$ fourth, respectively. The coefficients of the stresses in the posts and in the chords are calculated by algebraic resolution in the same manner as for a Baltimore truss.

*Live Load Stresses.*—The maximum stress in the diagonals in the third panel occurs with 7 loads to the right of the panel and with no loads to the left of the panel. The shear in the panel is then equal to the left reaction, equals $7 \times 4 \times P/10 = 14P/5$ and the coefficient of the stress is $+28$ twentieths and $-28$ twentieths in the upper and lower diagonal, respectively. The maximum shear in the fourth panel will occur with 6 loads to the right and no loads to the left, and will be equal to the left reaction, equals $6 \times 3\frac{1}{2} \times P/10 = 21P/10$, and the coefficient of the stress is $+21$ twentieths and $-21$ twentieths in the upper and lower diagonals, respectively. The maximum negative shear in the sixth panel occurs with 4 loads on the truss to the right and no loads to the left. The diagonals are designed to take a reversal of stress and no counters are required. The maximum stresses occur in the upper section of the posts when maximum stress occurs in the members meeting them on the top chord have a maximum stress. The maximum stresses in the lower part of the posts, with the exception of the second post which is really a tie, occur when maximum stress occurs in the diagonal member meeting the top of the lower section of the post.

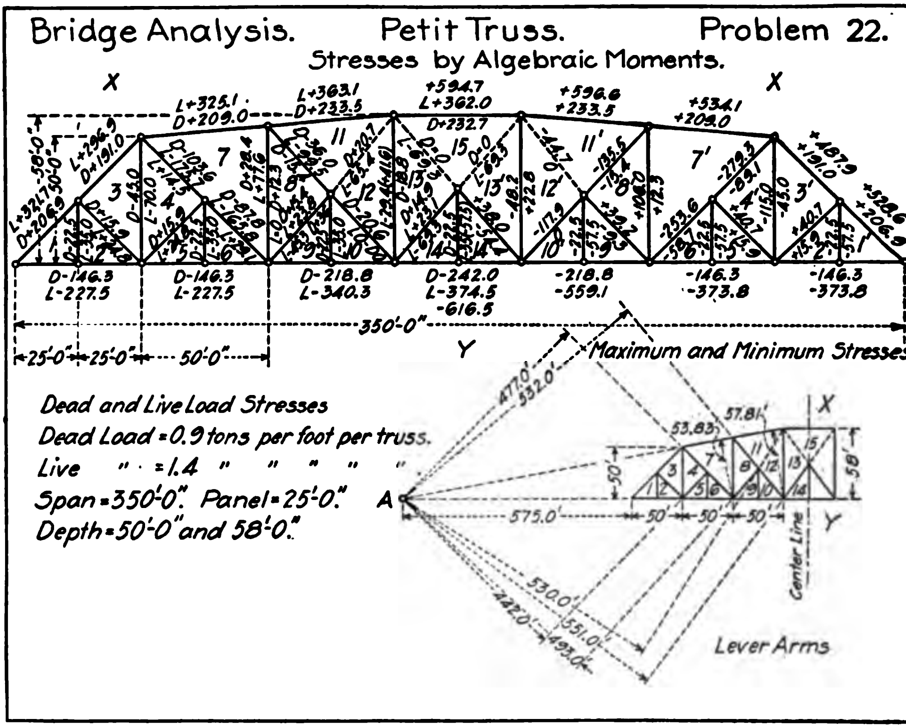
(a) **Results.**—It will be noted that the upper part of the K-truss has Howe truss diagonals and vertical ties, while the lower part has Pratt truss diagonals and vertical posts. The K-truss has smaller secondary stresses due to rigidity of the joints than either the Baltimore or Petit truss, and is rapidly replacing these trusses for long span bridges. For long spans the K-truss is commonly made with inclined upper chords.

PROBLEM 21A. MAXIMUM AND MINIMUM STRESSES IN A THROUGH K-TRUSS BY ALGEBRAIC RESOLUTION (METHOD OF COEFFICIENTS).

(c) **Problem.**—Given a through K-truss, span 216′ 0″, panel length 18′ 0″, depth 36′ 0″, dead load 5 tons per joint per truss, live load 6 tons per joint per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic resolution. Scale of truss 1″ = 30′ 0″.

PROBLEM 22. MAXIMUM AND MINIMUM STRESSES IN A PETIT TRUSS BY ALGEBRAIC MOMENTS.

(a) **Problem.**—Given a Petit truss, span 350′ 0″, panel length 25′ 0″, depth at the hip 50′ 0″, depth at center 58′ 0″, dead load 0.9 tons per lineal foot per truss, live load 1.4 tons per lineal foot per truss. Calculate the maximum and minimum stresses due to dead and live loads by algebraic moments. Scale of truss, 1″ = 40′ 0″. Scale of lever arms, any convenient scale.




(*b*) **Methods.**—Construct a truss diagram carefully to scale as shown. Construct one-half the truss to scale on a large piece of paper and calculate the lever arms as shown, and check by scaling from the diagram. The methods of calculation will be shown by two examples:

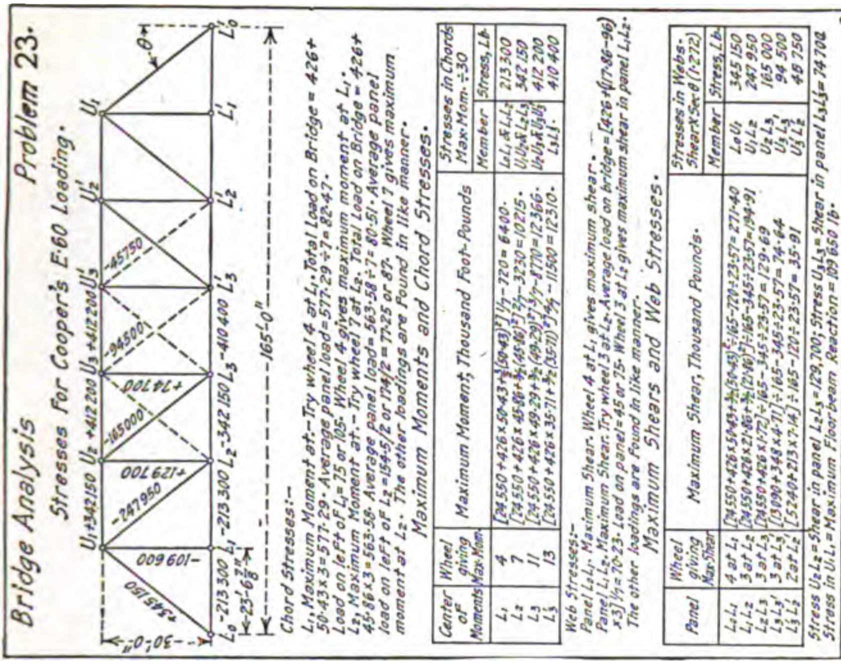
1. *Stresses in Tie* 6–7. *Dead Load Stress.*—Pass a section cutting members 7–$X$, 6–7, and 6–$Y$, and cutting away the truss to the right. The center of moments will be at $A$, the intersection of chords 7–$X$ and 6–$Y$. Now assume the stress in 6–7 as an external force acting from the outside toward the cut section. Then for equilibrium $6\text{–}7 \times 477.0 + R_1 \times 575 - 3W \times 625 = 0$. Now $R_1 = 146.25$ tons and $W = 22.5$ tons, and solving the equation gives stress $6\text{–}7 = -87.8$ tons.

*Live Load Stresses.*—The maximum live load stress in 6–7 will occur with the longer segment of the truss loaded. Taking moments about point $A$ as for the dead loads the maximum live load stress $6\text{–}7 \times 477.0 + R_1 \times 575 = 0$. Now $R_1 = 55/14 \times 35$ tons $= 137.5$ tons, and the stress in $6\text{–}7 = -165.8$ tons.

The minimum live load stress in 6–7 will occur with the shorter segment of the truss loaded. Taking moments about the point $A$, $6\text{–}7 \times 477.0 + R_1 \times 575 - 3P \times 625 = 0$. Now $R_1 = 90$ tons, $P = 35$ tons, and stress in $6\text{–}7 = +29.1$ tons.

2. *Stresses in Tie* 4–7. *Dead Load Stress.*—Pass a section cutting members 7–$X$, 4–7, 4–5 and 5–$Y$, and cutting away the truss to the right. Now assume the stress in 4–7 as an external force acting from the outside toward the cut section. Then for equilibrium about the point $A$, stress $4\text{–}7 \times 477.0 + R_1 \times 575 -$ stress $4\text{–}5 \times 442.0 - 2W \times 612.5 = 0$. Now the member 4–5 will carry one-half the load carried by 5–6, and the stress equals $1/2 \times 22.5 \times 1.414 = +15.9$ tons. $R_1 = 146.25$ tons, and $2W = 45$ tons. Then stress $4\text{–}7 = -103.6$ tons.

*Live Load Stresses.*—The maximum live load stress in 4–7 will occur with the longer segment loaded. Taking moments about $A$ as for dead loads, stress $4\text{–}7 \times 477.0 + R_1 \times 575 -$ stress




4–5 × 442.0 = 0. Now stress 4–5 = + 24.8 tons, and $R_1$ = 66/14 × 35 = 165 tons. Then stress 4–7 = − 175.7 tons.

The minimum live load stress in 4–7 will occur with two loads to the left of the panel. Taking moments about the point $A$, the stress 4–7 × 477.0 + $R_1$ × 575 − $2P$ × 612.5 = 0. Now $R_1$ = 62.5 tons and $2P$ = 70 tons. Then stress 4–7 = + 14.5 tons.

The stresses in the members in the first and second panels and in the two middle panels may be calculated by coefficients. Check up the dead load chord stresses by comparing with the the stresses obtained by graphic resolution in Problem 6.

(*c*) **Results.**—The auxiliary members carry the stresses directly toward the abutments and there is no ambiguity of loading as in the case of a truss subdivided as in Problem 16. However, the method of subdividing shown in Problem 16 is used in preference to that shown in this problem. The Petit truss is quite generally used for long span pin-connected highway and railway bridges.

PROBLEM 22A. MAXIMUM AND MINIMUM STRESSES IN A PETIT TRUSS BY ALGEBRAIC MOMENTS.

(*a*) **Problem.**—Given a Petit truss with the same span and loads as in Problem 22, the auxiliary bracing to be the same as in Problem 16. Calculate the maximum and minimum stresses due to dead and live loads by algebraic moments.

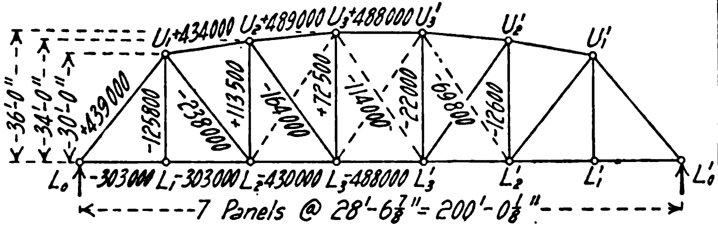
PROBLEM 23. LIVE LOAD STRESSES IN A THROUGH PRATT TRUSS FOR COOPER'S E 60 LOADING.

(*a*) **Problem.**—Given a Pratt truss, span 165′ 0″, panel length 23′ 6⅞″, depth 30′ 0″, live load Cooper's E 60 loading. Calculate the position of the loads and the maximum and minimum stresses due to the prescribed loading by algebraic moments. Scale of truss, 1″ = 25′ 0″.

(*b*) **Methods.** *Chord Stresses.*—Calculate the position of the wheels for a maximum bending moment at the different joints in the lower chord. The criterion for maximum bending moment

## Bridge Analysis — Problem 24

### Stresses for Cooper's E-60 Loading




### Chord Stresses

$L_1$, Maximum moment at:- Try wheel 4 at $L_1$. Total load on bridge = 426 + 80.44 × 3 = 667.32. Av. panel load = 667.32 ÷ 7 = 95.33. Load on left of $L_1$ = 75 or 105. Wheel 4 gives max. moment at $L_1$.
$L_2$, Maximum moment at:- Try wheel 8 at $L_2$. Total load on bridge = 426 + 76.87 × 3 = 656.61. Av. panel load = 656.61 ÷ 7 = 93.81. Av. panel load on left of $L_2 = \frac{174}{2}$ or $\frac{193.5}{2}$ = 87 or 96.75. Wheel 8 gives max. moment at $L_2$. The other loadings are found in like manner.

Maximum Moments and Chord Stresses

| Mom. center | Wheel | Max. Moment, Thousand Ft.-Lb. | Mom. Arm U | Mom. Arm L | Mem. | Stress |
|---|---|---|---|---|---|---|
| $L_1$ | 4 | $[24550 + 426 \times 80.44 + \frac{3}{2}(80.44)^2]\frac{1}{7} - 720 = 9090$ | | 30 | $L_0L_1$, $L_1L_2$ | 303000 |
| | | | 20.73 | | $L_0U_1$ | 439000 |
| $L_2$ | 8 | $[24550 + 426 \times 76.87 + \frac{3}{2}(76.87)^2]\frac{2}{7} - 4280 = 14619$ | | 34 | $L_2L_3$ | 430000 |
| | | | 33.67 | | $U_1U_2$ | 434000 |
| $L_3$ | 12 | $[24550 + 426 \times 74.28 + \frac{3}{2}(74.28)^2]\frac{3}{7} - 10060 = 17569$ | | 36 | $L_3L_3'$ | 488000 |
| | | | 35.91 | | $U_2U_3$ | 489000 |
| | | | 36. | | $U_3U_3'$ | 488000 |

### Web Stresses

Panel $L_0L_1$, Maximum Shear. Wheel 4 at $L_1$ gives max. shear.
Panel $L_1L_2$, Maximum Shear. Try wheel 3 at $L_2$. Av. load on bridge =



## Bridge Analysis — Problem 24 cont'd

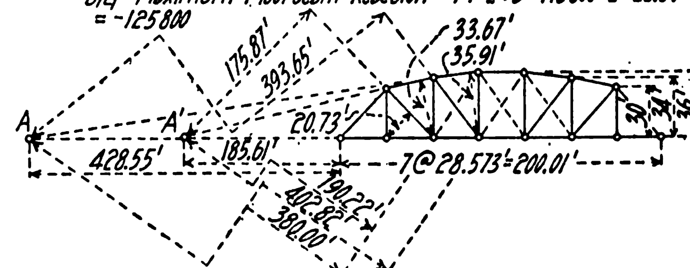
$[426 + 46.87 \times 3]\frac{1}{7} = 80.94$. Load on panel $= (45 \text{ or } 75)(1 + \frac{57.14}{185.61}) = 59$ or 98. Wheel 3 at $L_2$ gives maximum shear in panel $L_1L_2$. The other loadings are found in like manner.

Maximum Shears and Web Stresses

| Panel | Wheel | Maximum Shear, Thousand Lb. | Mem. | Stress |
|---|---|---|---|---|
| $L_0L_1$ | 4 at $L_1$ | $[24550 + 426 \times 80.44 + \frac{3}{2}(80.44)^2] \div 200 - 720 \div 28.57 = 317.55$ | $L_0U_1$ | 439000 |
| $L_1L_2$ | 3 at $L_2$ | $R_1 = [24550 + 426 \times 46.87 + \frac{3}{2}(46.87)^2] \div 200 = 239.25$ | | |
| | | (A) $[239.25 \times 185.61 - 12.07 \times 214.18] \div 175.87 = 238.00$ | $U_1L_2$ | 238000 |
| $L_2L_3$ | 3 at $L_3$ | $R_1 = [24550 + 426 \times 18.3 + \frac{3}{2}(18.3)^2] \div 200 = 164.25$ | | |
| | | (A) $[164.25 \times 428.55 - 12.07 \times 485.69] \div 393.65 = 164.00$ | $U_2L_3$ | 164000 |
| $L_3L_3'$ | 3 at $L_3'$ | $R_1 = [18060 + 387 \times 5.71] \div 200 = 101.40$   $345 \div 28.57 = 12.07$ | | |
| | | Shear $= 101.40 - 12.07 = 89.33$   $89.33 \times 1.276 = 114.00$ | $U_3L_3'$ | 114000 |
| $L_3'L_2'$ | 2 at $L_2'$ | $R_1 = [8770 + 258 \times 1.14] \div 200 = 45.32$   $120 \div 28.57 = 4.21$ | | |
| | | (A") $[45.32 \times 628.56 - 4.21 \times 514.26] \div 380.00 = 69.80$ | $U_3'L_2'$ | 69800 |

### Stresses in Vertical Posts

Stress $U_2L_2$ max. $[164.25 \times 185.61 \div 242.75 - 12.07] \times 1000 = +113500$
$U_2L_2$ min. $[185.61 \times 212.68 + 190.22 \times 69.8 - 53190) \div 242.75 - 10.79] \times 1000 = -12600$
$U_3L_3$ max. $[428.55 \times 101.40] \div 514.26 - 12.07] \times 1000 = +72500$
$U_3L_3$ min. $[(428.55 \times 285.60 + 402.82 \times 114.00 - 162640) \div 514.26 - 32.93] \times 1000 = -22000$
$U_1L_1$ = Maximum Floorbeam Reaction $= M \times 2 \div d = 178800 \times 2 \div 28.57 = -125800$



at any joint in a Pratt truss is, "the average load on the left of the section must be the same as the average load on the entire bridge." Having determined the wheel that is at the joint for a maximum moment, calculate the maximum bending moment as shown. Having calculated the maximum bending moments, the chord stresses are found by dividing the bending moment by the depth of the truss. The moment diagram is given in Table II, Chapter IV.

*Web Stresses.*—Calculate the position of the wheels for maximum shears in the different panels. The criterion for maximum shear in a panel is, "the load on the panel must equal the load on the bridge divided by the number of panels." The criterion for maximum bending moment at $L_1$ is the same as the criterion for maximum shear in panel $L_0L_1$. Having determined the position of the wheels for maximum shears in the different panels, calculate the maximum shears as shown. The stress in a web is equal to the shear in the panel multiplied by sec $\theta$.

*Floorbeam Reaction.*—The stress in the hip vertical $U_1L_1$ is equal to the maximum floorbeam reaction. The floorbeam reaction is equal to the bending moment at the center of a span equal to two panel lengths, multiplied by 2 divided by the panel length.

(*c*) **Results.**—When the maximum stresses occur in chords $U_2U_3$, $U_3U_3'$ and $L_3L_3'$, counter $U_3'L_3$ is in action. If more than one position of the loading will satisfy the criterion for maximum bending moment, the moments for each loading must be calculated.

PROBLEM 23A. LIVE LOAD STRESSES IN A THROUGH PRATT TRUSS FOR COOPER'S E 60 LOADING.

(*a*) **Problem.**—Given a Pratt truss, span 200′ 0″, panel length 25′ 0″, depth 32′ 0″, live load Cooper's E 60 loading. Calculate the position of the loads and the maximum and minimum stresses due to the prescribed loading by algebraic moments. Check by calculating maximum and minimum stresses for equivalent uniform live load as given in Fig. 3, Chapter XIX.

PROBLEM 24. LIVE LOAD STRESSES IN A CAMEL-BACK TRUSS FOR COOPER'S E60 LOADING.

(a) **Problem.**—Given a camel-back truss, span 200′ 0½″, panel length 28′ 6⅞″, depth at hip 30′ 0″, depth at second panel point 34′ 0″, depth at center 36′ 0″. Calculate the position of the loads and the maximum and minimum stresses due to the prescribed loading by algebraic moments.

(b) **Methods.** *Chord Stresses.*—Calculate the position of the wheels for maximum bending moment at the different joints in the lower chord. The criterion for the maximum bending moment at any joint in a camel back truss is, "the average load on the left of the section must be the same as the average load on the entire bridge." Having determined the wheel that is at the joint for a maximum moment, calculate the maximum bending moment as shown. Having calculated the maximum bending moments, the upper chord stresses are found by dividing the bending moment by the perpendicular distance from the joint to the member. The lower chord stresses are found by dividing the bending moment by the depth of the truss. The moment diagram is given in Table II, Chapter IV.

*Web Stresses.*—Calculate the position of the wheels for maximum shears in the different panels. The criterion for maximum shear in a panel of a truss with inclined chords is formula (20) Chapter IV. The criterion for maximum bending moment at $L_1$ is the same as the criterion for maximum shear in panel $L_0L_1$. Having determined the position of the wheels for maximum shears in the different panels, calculate the shears and stresses as shown.

(c) **Results.**—When the maximum stresses occur in chords $U_2U_3$, $U_3U_3'$ and $L_3L_3'$, counter $U_3'L_3$ is in action. If more than one position of the loading will satisfy the criterion for maximum bending moment, the moments for each loading must be calculated.

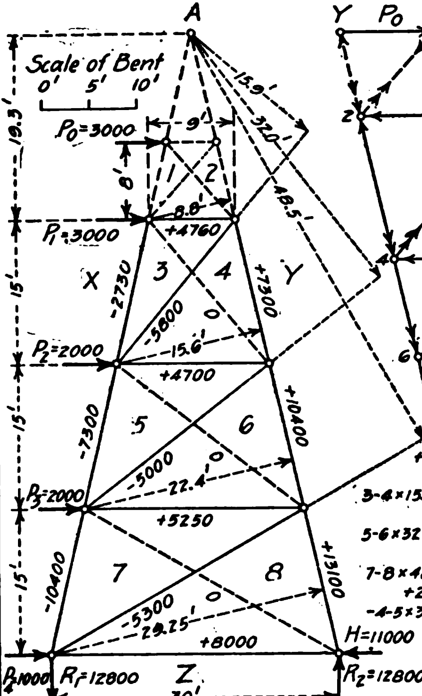
PROBLEM 24A. LIVE LOAD STRESSES IN A CAMEL-BACK TRUSS FOR COOPER'S E60 LOADING.

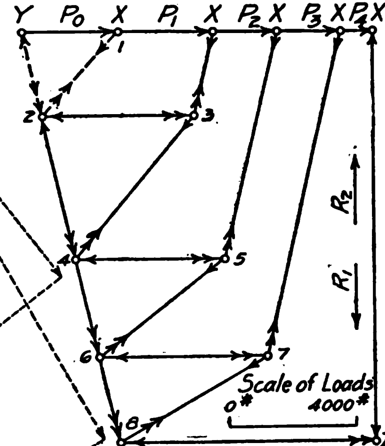
(a) **Problem.**—Given a camel-back truss, span 200′ 0″, panel length 25′ 0″, depth at hip 30′ 0″, depth at second panel 34′ 0″, depth at third and center panel 36′ 0″. Calculate the position of the loads and the maximum and minimum stresses by algebraic moments.

8

# Bridges Analysis. Problem 26.

## Wind Load Stresses-Trestle Bent.




$+R_1\times30-2000\times15-2000\times30-3000\times45$
$-3000\times53=0 \quad R_1=R_2=12800$

$3\text{-}4\times15.9+3000\times19.3+3000\times11.3=0$
$3\text{-}4=-5800$

$5\text{-}6\times32+3000\times19.3+3000\times11.3+2000\times34.3=0$
$5\text{-}6=-5000$

$7\text{-}8\times48.5+3000\times19.3+3000\times11.3+2000\times34.3$
$+2000\times49.3=0 \quad 7\text{-}8=-5300$

$-4\text{-}5\times34.3+3000\times19.3+3000\times11.3$
$+2000\times34.3=0 \quad 4\text{-}5=+4700$

$-6\text{-}7\times49.3+3000\times19.3+3000\times11.3$
$+2000\times34.3+2000\times49.3=0$
$6\text{-}7=+5250$

$(2\text{-}3-3000)\times19.3+3000\times11.3=0$
$2\text{-}3=+4760$

$-8\text{-}Z\times64.3+3000\times19.3+3000\times11.3$
$+2000\times34.3+2000\times34.3+2000\times49.3$
$+1000\times64.3=0 \quad 8\text{-}Z=+8000$

$3\text{-}X\times8.8-3000\times8=0 \quad 3\text{-}X=-2730$

$-3000\times23-3000\times15+4\text{-}Y\times15.6=0$
$4\text{-}Y=+7300=-5\text{-}X$

$6\text{-}Y\times22.4-3000\times38-3000\times30-2000\times15=0$
$6\text{-}Y=+10400=-7\text{-}X$

$8\text{-}Y\times29.25-3000\times53-3000\times45-2000\times15-2000\times30=0 \quad 8\text{-}Y=+13100$



# Bridge Analysis Problem 25.

## Stresses in a Portal.

(8) $V=\dfrac{30\times2000}{15}=4000$

(A) $V'=-\dfrac{30\times2000}{15}=-4000$

(1) $3\text{-}Y=\dfrac{-30\times1000}{10\times0.447}=-6710$

(2) $4\text{-}b=\dfrac{2000\times5+1000\times25-V'\times2.5}{5}$
$=+5000$

(3) $3\text{-}4=\dfrac{(2000-5000)\times10+1000\times20}{4.47}$
$=-2270$

(4) $4\text{-}5=\dfrac{5000\times5+1000\times25-4000\times7.5}{5\times0.894}$
$=-4470$

(5) $5\text{-}Y=\dfrac{1000\times30-4000\times5}{5}$
$=-2000$

(4) $6\text{-}b=\dfrac{1000\times25-4000\times7.5+2000\times5}{5}=+1000$

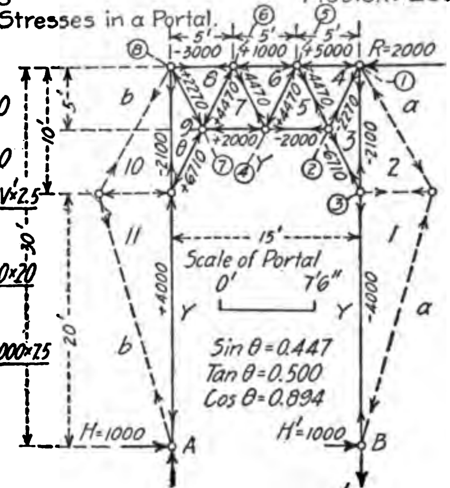
(6) $7\text{-}Y=\dfrac{1000\times30-4000\times10}{5}=+2000$

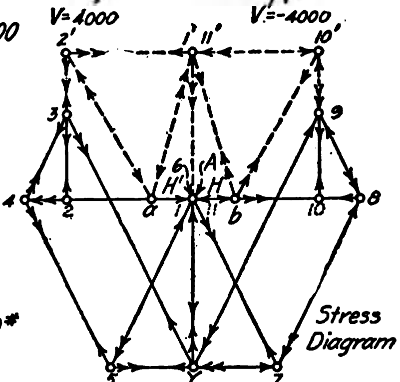
(7) $8\text{-}b=\dfrac{1000\times25-4000\times2.5}{5}=-3000$

(8) $9\text{-}Y=\dfrac{1000\times30}{10\times0.447}=+6710$

(6) $8\text{-}9=\dfrac{4000\times10-1000\times30}{10\times0.447}$
$=2270$

Scale of Loads
0# 2000# 4000#



### Problem 25. Stresses in the Portal of a Bridge by Algebraic Moments and Graphic Resolution.

(*a*) **Problem.**—Given the portal of a bridge of the type shown, inclined height 30′ 0″, center to center width 15′ 0″, load $R = 2,000$ lb., end-posts pin-connected at the base. Calculate the stresses by algebraic moments and check by graphic resolution. Scales as shown.

(*b*) **Methods.**—Now $H = H' = 1,000$ lb. $V = -V'$, and by taking moments about $B$, $V = 30 \times 2,000/15 = 4,000$ lb. $= -V'$.

*Algebraic Moments.*—In passing sections, care should be used to avoid cutting the end-posts for the reason that these members are subject to bending stresses in addition to the direct stresses. To calculate the stress in member $3-Y$ take the center of moments at joint (1) and pass a section cutting members $4-b$, $3-4$ and $3-Y$, and cutting the portal away to the left of the section. Then assume stress $3-Y$ as an external force acting from the outside toward the cut section, and $3-Y \times 10 \times 0.447 (\sin \theta) + H \times 30' = 0$. The stress in $3-Y = -6,710$ lb. The remaining stresses are calculated as shown

*Graphic Resolution.*—Lay off $a-A = A-b = H = 1,000$ lb., and $A-Y = V' = 4,000$ lb. Then beginning at point $B$ in the portal the force polygon for equilibrium is $a-A-Y-1'-a$, in which $1'-a$ is the stress in the auxiliary member $1-a$, and $Y-1'$ is the stress in the post $1-Y$ when the auxiliary member is acting. The true stress in $1-Y$ is equal to the algebraic sum of the vertical components of the stress $1'-a$ and $Y-1'$, and equals $V' = -4,000$ lb. Next complete the force triangle at the intersection of the auxiliary members. Stress $1'-a$ is known and the force polygon is $a-1'-2'-a$, the forces acting as shown. The stress diagram is carried through in the order shown, checking up at the point $A$. The correct stresses are shown by the full lines in the stress diagram. The true stress in $3-2$ will produce equilibrium for vertical stresses at joint (1) as shown. The maximum shear in the posts is $H = 1,000$ lb. The maximum bending moment in the posts will occur at the foot of the member $3-Y$, joint (3), and is $M = 1,000 \times 20 \times 12 = 240,000$ in.-lb.

(*c*) **Results.**—The method of graphic resolution requires less work and is more simple than the method of algebraic moments.

Note: The portal is not pin-connected at joints (3) and the corresponding joint on the opposite side, as might be inferred from the figure.

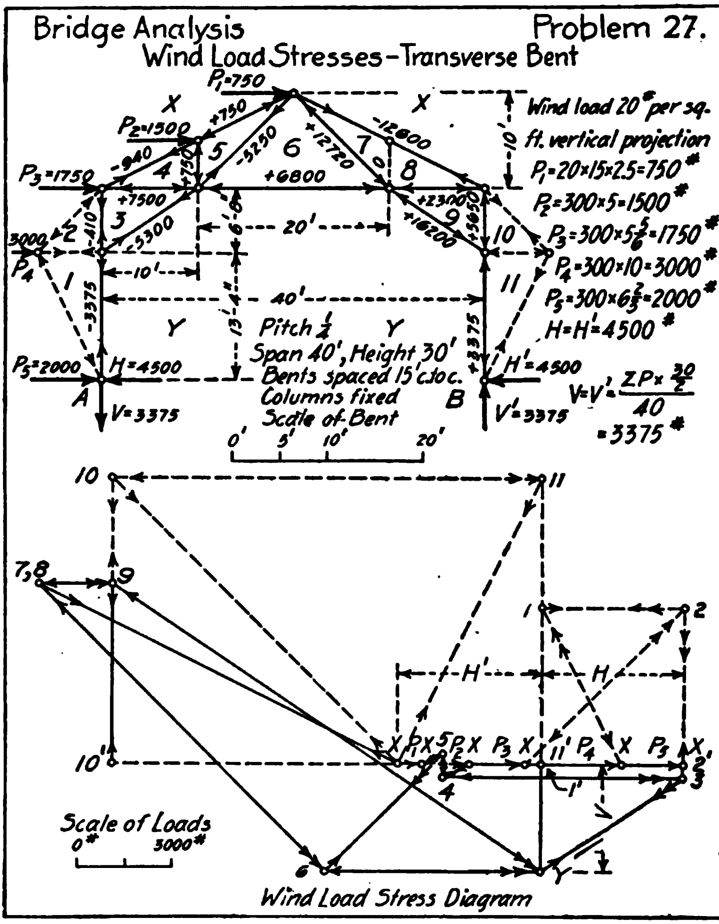
### Problem 25A. Stresses in the Portal of a Bridge by Algebraic Moments and Graphic Resolution.

(*a*) **Problem.**—Given the portal shown in Problem 25, except that the posts are fixed at their bases. Calculate the stresses by algebraic moments and check by graphic resolution. Assume the point of contraflexure as half way between the base of the post and the foot of the knee brace. Scales as in Problem 25.

### Problem 26. Wind Load Stresses in a Trestle Bent.

(*a*) **Problem.**—Given a trestle bent, height 45′ 0″, width at the base 30′ 0″, width at the top 9′ 0″, wind loads $P_0$, $P_1$, $P_2$, $P_3$, $P_4$, as shown. Calculate the stresses in the members of the bent due to wind loads by algebraic moments, and check by calculating the stresses by graphic resolution. Assume that the diagonal members are tension members, and that the dotted members are not acting for the wind blowing as shown. Scale of truss, 1″ = 10′ 0″. Scale of loads, 1″ = 2,000 lb.

(*b*) **Methods.** *Algebraic Moments.*—To calculate the stresses in the diagonal members take centers of moments about the point $A$, the point of intersection of the inclined posts. Then to calculate the stress in $3-4$, pass a section cutting members $3-X$, $3-4$ and $4-Y$; assume that the stress in $3-4$ is an external force acting from the outside toward the cut section, and $3-4 \times 15.9' + 3,000 \times 19.3' + 3,000 \times 11.3' = 0$. The stress $3-4 = -5,800$ lb. Stresses in $4-5$, $5-6$, $6-7$, $7-8$ and $8-Z$ are calculated in a similar manner. To obtain reaction $R_1$ take moments about




$R_2$, and $R_1 \times 30' - 2{,}000 \times 15' - 2{,}000 \times 30' - 3{,}000 \times 45' - 3{,}000 \times 53' = 0$. Then $R_1 =$ 12,800 lb. $= - R_2$.

To calculate the stress in $4-Y$, take center of moments at joint $P_2$, and pass a section cutting members $5-X$, $4-5$ and $4-Y$, and assume the stress in $4-Y$ as an external force acting from the outside toward the cut section. Then $4-Y \times 15.6' - 3{,}000 \times 15' - 3{,}000 \times 23' = 0$. Then $4-Y = +\ 7{,}300$ lb.

*Graphic Resolution.*—The load $P_0$ is assumed as transferred to the bent by means of the auxiliary members. The loads $P_0$, $P_1$, $P_2$, $P_3$, $P_4$ are laid off as shown, and with the load $P_0$ the stress triangle $Y-X-2$ is drawn. The remainder of the solution is easily followed.

(*c*) **Results.**—The stress in the auxiliary member $2-Y$ acts as a load at the top of post $4-Y$. Load $P_0$ is the wind load on the train and is transferred to the rails by the car. For the reason that the wind may blow from the opposite direction, both sets of stresses must be considered in combination with the dead and live load stresses in designing the columns.

PROBLEM 26A. WIND LOAD STRESSES IN A TRESTLE BENT.

(a) **Problem.**—Given a trestle bent, height 54′ 0″, panels 18′ 0″, width at the base 30′ 0″, width at the top 8′ 0″, wind loads $P_0$, $P_1$, $P_2$, $P_3$, $P_4$ as shown in Problem 26. Calculate the stresses in the members of the bent due to wind loads by algebraic moments, and check by calculating the stresses by graphic resolution. Assume that the diagonal members are tension members, and that the dotted members are not acting for the wind blowing as shown. Scale of truss, 1″ = 10′ 0″. Scale of loads, 1″ = 2,000 lb.

PROBLEM 27. WIND LOAD STRESSES IN A TRANSVERSE BENT BY GRAPHIC RESOLUTION.

(a) **Problem.**—Given a transverse bent, span 40′ 0″, pitch of roof $\frac{1}{4}$, height of posts 20′ 0″, posts pin-connected at the base, wind load 20 lb. per square foot of vertical projection. Calculate the wind load stresses in the bent by graphic resolution. Scale of bent, 1″ = 10′ 0″. Scale of loads, 1″ = 3,000 lb.

(b) **Methods.**—Now $H = \frac{1}{2}\Sigma P$ = 4,500 lb. = $H'$. To calculate $V$ take moments about the foot of the right-hand post, and $V \times 40' - 3{,}000 \times 13\frac{1}{3}' - 1{,}750 \times 20' - 1{,}500 \times 25' - 750 \times 30' = 0$. Then $V = +\ 3{,}375$ lb. $= -\ V'$.

To construct the stress diagram lay off the load line $P_1 + P_2 + P_3 + P_4 + P_5$, and 1–$Y$ = $V$ = 3,375 lb. Beginning at the foot of the windward post, $V$ acts downward, $H$ = $X$–1 acts to the left, $P_5$ acts to the right. The polygon is closed by drawing lines parallel to 1–$X$ and 1–$Y$, the final stress polygon being $Y$–1–$X$–$X$–1′. Then pass to the load $P_4$ in the transverse bent, and in the stress diagram $P_4$ acts to the right, 1–$X$ acts upwards to the left, 1–2 acts to the right, and 2–$X$ acts downwards to the left, closing the polygon. The remainder of the stress diagram is drawn in a similar manner, passing to the foot of the knee brace, then to the top of the post, etc., finally checking up at the foot of the leeward post. The maximum shear is in the leeward post, below the knee brace the shear is $H$ = 4,500 lb., above the knee brace the shear is the horizontal component of the stress in 10–$X$ = 10′–$X$ = 9,000 lb. The maximum bending moment in the post is at the foot of the leeward knee brace and is $M = 4{,}500 \times 13\frac{1}{3} = 60{,}000$ ft.-lb. For further explanation see the author's "The Design of Steel Mill Buildings."

(c) **Results.**—The stresses in the members do not follow the usual rules for trusses loaded with vertical loads; the top chord is partly in tension and partly in compression, while the bottom chord is in compression. The bent should be designed for the wind load stresses combined with the dead load and the minimum snow load stresses, for the snow load and the dead load stresses, or for the wind load and the dead load stresses, whichever combination produces maximum stresses or reversals of stresses.

The stresses in the posts are calculated by dropping the points 1, 2, 10 and 11 to the points 1′, 2′, 10 and 11′, respectively, on the load line, or on load line produced. The stresses in the windward post are 1′–$Y$ and 2′–3, while the stresses in the leeward post are 11′–$Y$ and 9–10′. The maximum shear in the leeward post is above the knee brace and is 10′–$X$ = 9,000 lb.

PROBLEM 27A. WIND LOAD STRESSES IN A TRANSVERSE BENT BY GRAPHIC RESOLUTION.

(a) **Problem.**—Given a transverse bent, span 40′ 0″, pitch of roof $\frac{1}{4}$, height of posts 20′ 0″, posts pin-connected at the base, wind load 20 lb. per square foot normal to the sides and the normal component of a horizontal wind load of 30 lb. per square foot on the roof. (The normal load on the roof for a horizontal wind load of 30 lb., is 22½ lb. per sq. ft., see "Steel Mill Buildings.") Calculate the wind load stresses in the transverse bent by graphic resolution. Scale of bent, 1″ = 10′ 0″. Scale of loads, 1″ = 3,000 lb.

# PART II.

# DESIGN OF STEEL AND TIMBER BRIDGES.

## CHAPTER VIII.

### TYPES OF BRIDGES.

**Introduction.**—A truss is a framework composed of individual members so fastened together that loads applied at the joints produce only direct tension or compression. The triangle is the only geometrical figure in which the form is changed only by changing the lengths of the sides. In its simplest form every truss is a triangle or a combination of triangles. The members of the truss are either fastened together with pins, pin-connected, or with plates and rivets, riveted.

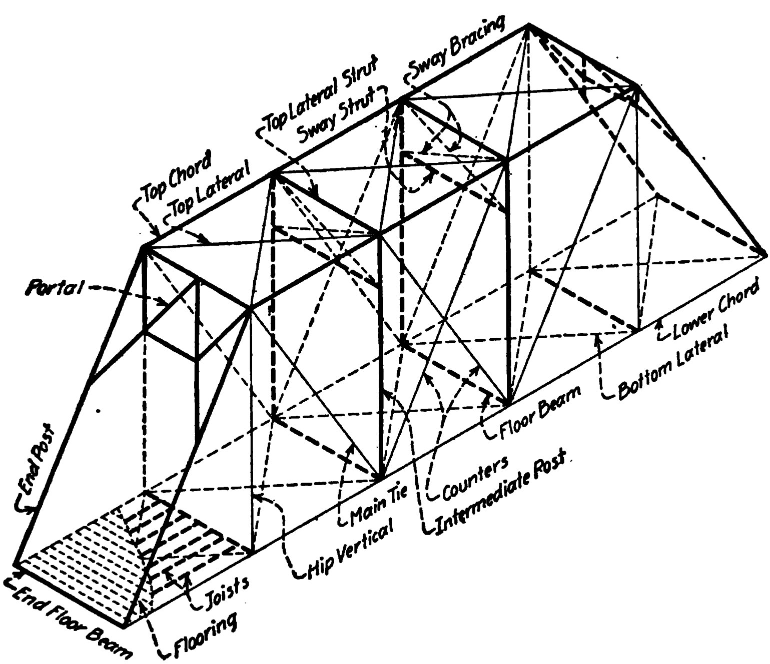



FIG. 1. DIAGRAMMATIC SKETCH OF A THROUGH PRATT TRUSS HIGHWAY BRIDGE.

The bridge in Fig. 1 consists of two vertical trusses which carry the floor and the load; two horizontal trusses in the planes of the top and bottom chords, respectively, which carry the horizontal wind load along the bridge; and cross-bracing in the plane of the end-posts, called

portals, and in the plane of the intermediate posts, called sway bracing. The floor is carried on joists placed parallel to the length of the bridge, and which are supported in turn by the floorbeams. The names of the different parts of the bridge are shown in Fig. 1. The main ties, hip verticals, counters and intermediate posts are together called webs. The bridge shown in Fig. 1 is a through pin-connected bridge of the Pratt type, the traffic passing through the bridge. The bridge shown

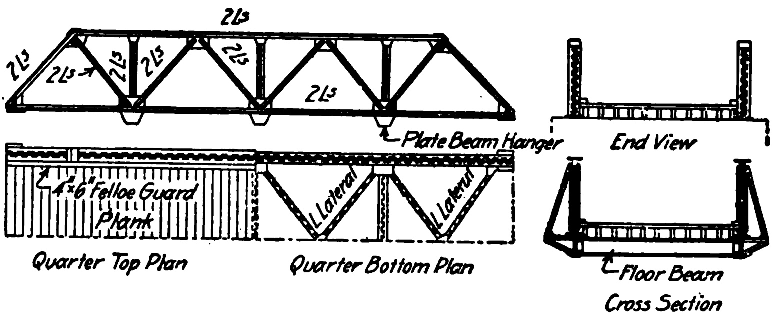



FIG. 2. PLAN OF A LOW OR "PONY" TRUSS HIGHWAY BRIDGE.

in Fig. 1 has square abutments; the abutments are not at right angles to the center line of the bridge in a "skew" bridge. Short span highway and railway bridges have low trusses and no top lateral system nor portals. In a railway bridge the track and ties are supported on stringers, which replace the joists in Fig. 1.

FIG. 3. A WARREN LOW TRUSS HIGHWAY BRIDGE.

A low truss highway bridge of the Warren type is shown in Fig. 2, and a view of a similar bridge is shown in Fig. 3. The trusses are built up of angles riveted together by means of connection plates. Bridges of this type are built with spans of from 30 to 75 feet. Low truss bridges are also made with pin-connected joints. A pin-connected low Pratt truss bridge is shown in Fig. 4.

FIG. 4. A PRATT LOW TRUSS HIGHWAY BRIDGE: SEVEN 100-FT. SPANS.

The loads are sometimes carried on the top chord as in Fig. 5, which is a highway bridge built for the U. S. Government in the Yellowstone Park. In this truss the end-posts, top chords and intermediate posts are composed of 2 channels laced; while the lower chords, hip verticals,

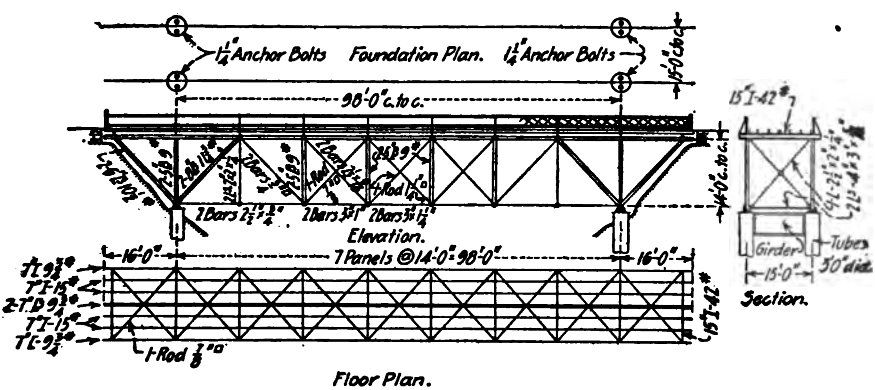



FIG. 5. DECK PRATT PIN-CONNECTED HIGHWAY BRIDGE.

main ties and counters are composed of eye-bars. The floorbeams are I beams 15 inches deep weighing 50 lb. per lineal foot (15″ I @ 50 lb.), while the joists are 7″ Is and 7″ [s. A deck highway bridge is shown in Fig. 6.

**Types of Trusses and Bridges.**—The simplest type of bridge is the beam bridge, (*a*) Fig. 7. Beam bridges commonly consist of I beams which span the opening, and are placed near enough

FIG. 6. DECK HIGHWAY BRIDGE.

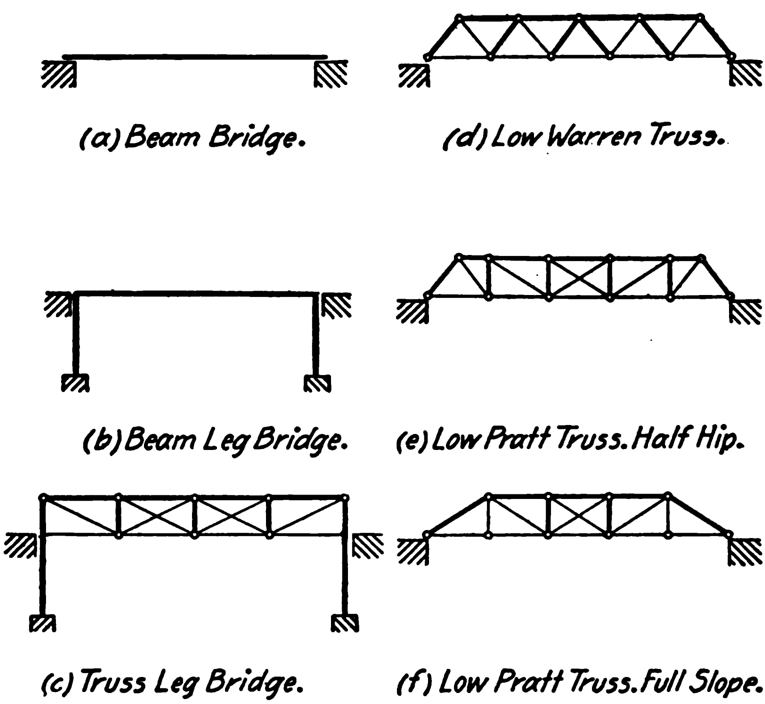



FIG. 7. TYPES OF SHORT SPAN HIGHWAY BRIDGES.

together to carry the floor of the bridge. Where foundations are relatively expensive the beams may be carried on posts as in (*b*) Fig. 7. A truss leg bridge is shown in (*c*) Fig. 7. Types (*b*) and (*c*) unless constructed with great care make inferior structures and are not to be recommended. A Warren truss is a combination of isosceles triangles as shown in (*d*) Fig. 7 and in (*c*) Fig. 8. The Pratt truss has its vertical web members in compression while its diagonal web members are in tension, as shown in (*e*) and (*f*) Fig. 7 and in (*b*) Fig. 8. The Warren truss is commonly built with riveted joints while the Pratt truss is usually built with pin-connected joints. The Warren low truss with riveted joints as shown in (*d*) is generally preferred in place of the low

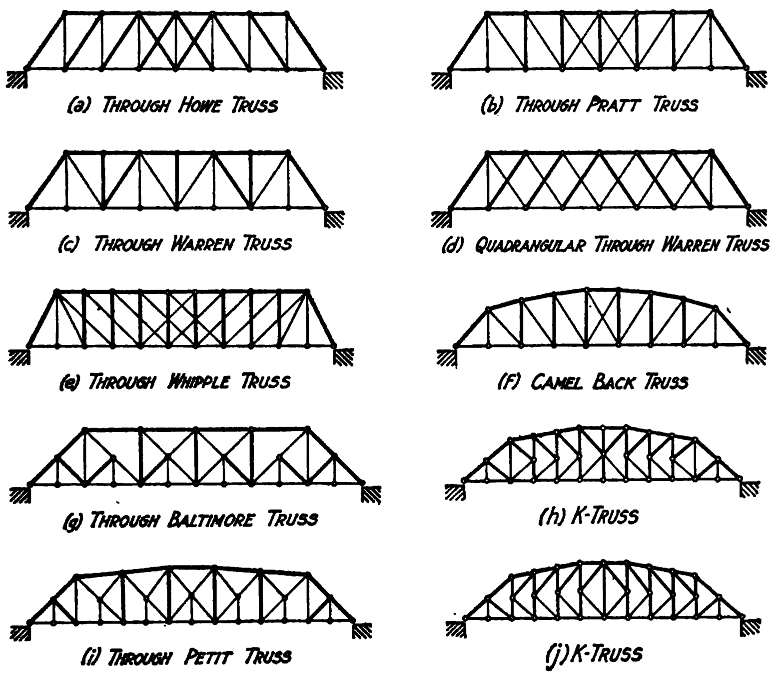



FIG. 8. TYPES OF HIGH TRUSS BRIDGES.

Pratt truss in either (*e*) or (*f*) Fig. 7. The Howe truss has its vertical web members in tension, and its inclined web members in compression as shown in (*a*) Fig. 8. The upper and lower chords and the inclined members of a Howe truss are commonly made of timber, while the vertical tension members are iron or steel rods.

The Whipple truss, (*e*), Fig. 8, is a double intersection Pratt truss. This truss was designed to give short panels in long spans which have a considerable depth. The stresses in the Whipple truss are indeterminate for moving loads, and its use has been practically abandoned, the Baltimore truss, (*g*), Fig. 8, being used in its place. The quadrangular Warren truss, (*d*), Fig. 8., and Fig. 10, with riveted joints, is used as a standard truss for through highway bridges, with spans of from 80 to 170 feet, by the American Bridge Company. Like the Whipple truss its stresses are indeterminate for moving loads.

For spans of from, say, 170 to 240 feet it is quite common to use pin-connected trusses of the Pratt type having inclined chords as in (*f*), Fig. 8, and Fig. 11.

The Baltimore truss, (*g*), Fig. 8, is a Pratt truss with parallel chords in which the main panels have been subdivided by an auxiliary framework. The auxiliary framework may have struts

as in (*g*), or ties as in (*i*), Fig. 8. The Baltimore truss with inclined upper chords, (*i*), **Fig. 8**, is called a Petit truss. The stresses in Baltimore and Petit trusses are statically determinate for all conditions of loading. These trusses are economical in construction and satisfactory in service, and have entirely replaced the Whipple truss for long span bridges.

FIG. 9. THROUGH RIVETED PRATT TRUSS, 111′ 6″ SPAN, OVER ILLINOIS AND MISSISSIPPI CANAL.

FIG. 10. THROUGH RIVETED QUADRANGULAR WARREN TRUSS, BUILT BY BOSTON BRIDGE WORKS.

The K-truss shown in (*h*) and (*j*), Fig. 8, is more economical than the Petit truss, and in addition has smaller secondary stresses, and is rapidly coming into general use.

The types of simple bridge trusses described above are those that are in the most common use, although quite a number of other types of trusses have been used and abandoned.

FIG. 11. PARKER OR CAMELS-BACK, PIN-CONNECTED HIGHWAY BRIDGE, BUILT BY AMERICAN BRIDGE COMPANY.

FIG. 12. PLATE GIRDER HIGHWAY BRIDGE, BUILT BY AMERICAN BRIDGE COMPANY.

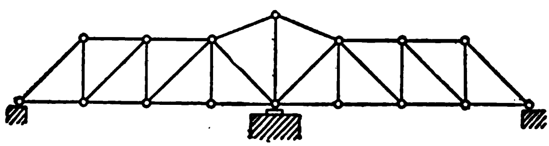


FIG. 13. SWING BRIDGE, CENTER BEARING

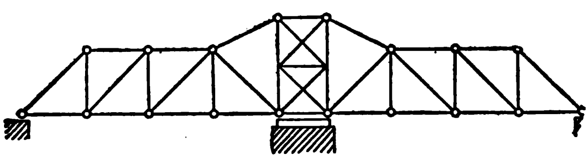


FIG. 14. SWING BRIDGE, TURNTABLE BEARING.

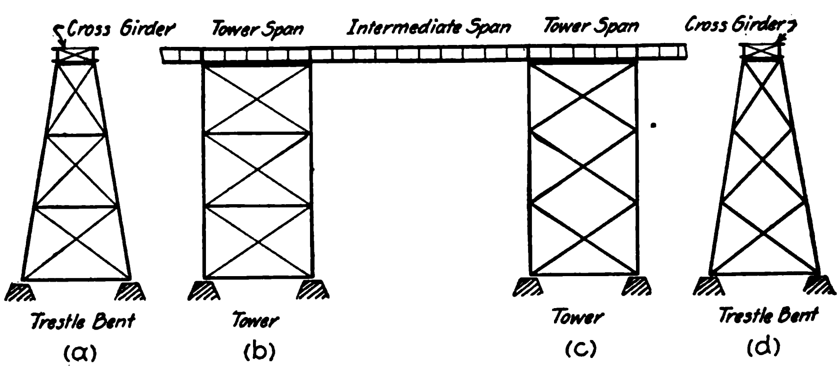



FIG. 15. RAILWAY STEEL TRESTLE.

**BEAMS AND PLATE GIRDERS.**—For spans of, say, 30 feet and under rolled beams are often used to carry the roadway, while for spans from about 30 to 100 feet plate girders are used. When the roadway is carried on top of the girders, the bridge is called a deck plate girder bridge, and when the roadway passes between the girders, the bridge is called a through plate girder bridge as in Fig. 12.

FIG. 16. CLARITON-CLIFTON TWO-HINGED ARCH HIGHWAY BRIDGE OVER NIAGARA RIVER.

**SWING BRIDGES.**—Swing bridges may be made of plate girders or trusses, and may turn on a center pivot as in Fig. 13, or on a turntable supported on a drum as in Fig. 14. The center pivot swing bridge has two spans continuous over the pivot support, while the turntable swing

FIG. 17. CANTILEVER HIGHWAY BRIDGE.

bridge has three spans ordinarily continuous over the two turntable supports. When the swing bridge is open each arm acts as a simple cantilever span.

**STEEL TRESTLES.**—Steel trestles are used for carrying the roadway at a considerable distance above the ground, Fig. 15. The tower and intermediate spans are commonly built of

plate girders, whether the trestle carries a railroad or a highway roadway. The tower consists of two trestle bents as in (*a*) or (*d*), braced together by longitudinal bracing as in (*b*) or (*c*) Fig. 15. Bracing as in (*a*) and (*b*) is used with either adjustable or rigid diagonal members, while bracing (*c*) and (*d*) is used only for rigid members.

**STEEL ARCHES.**—Steel arch bridges are made (1) with three hinges, (2) with two hinges, and (3) without hinges, and may have solid webs, or spandrel or open webs. A two-hinged highway arch is shown in Fig. 16.

FIG. 18. SUSPENSION HIGHWAY BRIDGE OVER NIAGARA RIVER AT QUEENSTON, ONTARIO.

**CANTILEVER BRIDGES.**—A cantilever bridge consists of two anchor spans, which support a suspended or channel span. The shore ends of the anchor spans are anchored to the shore pier and are supported on the river pier. A cantilever highway bridge is shown in Fig. 17.

**SUSPENSION BRIDGES.**—In a suspension bridge the roadway is supported by hangers attached to the main cables. Stiffening trusses are placed above the plane of the roadway to assist in distributing the live loads and for the purpose of increasing the rigidity of the structure. The suspension highway bridge over the Niagara River at Queenston, Ont., is shown in Fig. 18.

Simple truss bridges, beam and plate girder bridges, only, will be considered in this book.

# CHAPTER IX.

## Data for the Design of Steel Highway Bridges.

**TYPES OF STRUCTURE.**—The types of structure for steel highway bridges as recommended by the author are given in section 3, "General Specifications for Steel Highway Bridges," printed in Appendix I.

The following data will show present standard practice.

**Illinois Highway Commission.**—The types of highway bridge recommended by the commission are as follows:

*Concrete Bridges.*—For culverts requiring a waterway of 12 square feet or less, plain or reinforced concrete arch culverts or square culverts, reinforced concrete pipes or double strength cast-iron pipe.

For culverts having an area of more than 12 square feet, and for bridges having a span up to 30 ft., reinforced concrete slabs, plain or reinforced concrete arches.

For spans of 30 ft. to 65 ft., reinforced concrete through or deck girders, plain or reinforced concrete arches.

For spans greater than 65 ft., plain or reinforced concrete arches.

*Steel Bridges.*—For spans of 12 ft. to 45 ft., steel I-beams; for spans of 30 ft. to 100 ft., plate girders or riveted pony trusses; for spans of 90 ft. to 160 ft., riveted trusses with parallel chords; for spans of 150 ft. and more, riveted or pin-connected trusses with parallel or inclined upper chords.

**Iowa Highway Commission.**—The types of highway bridges recommended by the commission are as follows:

*Concrete Bridges.*—Box culverts for spans up to 16 ft.; slab bridges for spans from 14 ft. to 25 ft.; arch culverts and bridges for spans of 6 ft. and over; girder bridges for spans of from 24 ft. to 40 ft.

*Steel Bridges.*—Steel I-beams up to 32 ft. span; plate girders, 20 ft. to 80 ft. span; low truss 30 ft. to 100 ft. span; high truss 100 ft. span and over, riveted up to 140 ft. span.

**Massachusetts Public Service Commission.**—The types of highway bridge recommended by the commission are as follows:

*Steel Bridges.*—For spans up to 20 ft., wooden stringers or rolled beams; for spans from 20 ft. to 40 ft., rolled beams or plate girders; for spans from 40 ft. to 70 ft., plate girders; for spans from 70 ft. to 100 ft., plate girders or riveted trusses; for spans from 100 ft. to 125 ft., riveted trusses; for spans from 125 ft. up, riveted or pin trusses.

**Wisconsin Highway Commission.**—The types of highway bridge recommended by the commission are as follows:

*Concrete Bridges.*—Spans of $1\frac{1}{2}$ ft. to 10 ft., slab culverts and bridges; spans 10 ft. to 18 ft., slab bridges; spans 10 ft. to 40 ft., through girders.

*Steel Bridges.*—Spans 10 ft. to 38 ft., rolled beams; spans 35 ft. to 80 ft., Warren riveted low trusses or plate girders; spans 80 ft. to 135 ft., Pratt riveted high trusses; spans over 135 ft., riveted high trusses with curved chords.

**WIDTH OF ROADWAY.**—The following data will show standard practice.

**Illinois Highway Commission.**—The widths of roadways are specified for State Aid Routes, Principally Traveled Roads, and Secondary Roads.

*On Designated State Aid Routes.*—Bridges up to and including 10 ft. span, 20 to 30 ft. roadway; bridges over 10 ft. up to and including 60 ft. span, 18 to 24 ft. roadway; bridges over 60 ft. span, 16 to 20 ft. roadway.

*On Principally Traveled Roads.*—Bridges and culverts 10 ft. or less in span, 20 to 30 ft. roadway; bridges over 10 ft. and up to and including 60 ft. span, 16 to 20 ft. roadway· bridges over 60 ft. span, 16 to 18 ft. roadway.

*On Secondary Roads.*—Bridges and culverts 10 ft. or less in span, 18 to 24 ft. roadway; bridges over 10 ft. span, 16 ft. roadway.

*Culverts Under Fills.*—The barrel of the culvert shall have a .ength that will permit of side slopes of 1½ horizontal to 1 vertical, and a top width of 20 to 30 ft. on State Aid Routes, 20 to 30 ft. on Principally Traveled Roads, and 18 to 24 ft. on Secondary Roads.

**Iowa Highway Commission.**—The widths of roadway for highway bridges as recommended by the commission are as follows:

*Concrete Bridges.*—For box or arch culverts with spans of 2 ft. to 16 ft., 24 ft. roadway for county roads, and 20 ft. for township roads; for slab bridges with spans over 16 ft. span, 20 ft. roadway for county roads, and 18 ft. for township roads; for girder bridges over 16 ft. span, 20 ft. roadway; for arches over 16 ft. span, 24 ft. roadway for county roads, and 20 ft. for township roads. The slopes on fills shall be 1½ horizontal to 1 vertical.

*Steel Bridges.*—A roadway of 20 ft. on county roads, for all spans, and 18 ft. on township roads for all spans. The minimum legal width of roadway is 16 ft.

**Association of State Highway Departments.**—The following minimum widths of concrete bridges are recommended.

*For First Class Roads.*—Culverts under 12 ft. span, 24 ft. roadway; slab bridges over 12 ft. span, 20 ft. roadway; all other spans 20 ft. roadway.

*For Second Class Roads.*—Culverts under 12 ft. span, 20 ft. roadway; slab bridges over 12 ft. span, 18 ft. roadway; all other spans, 18 ft. roadway.

*For Third Class Roads.*—Culverts under 12 ft. span, 20 ft. roadway; slab bridges over 12 ft. span, 18 ft. roadway; longer bridges, 16 ft. roadway.

The above widths of concrete bridges have been adopted by the Wisconsin Highway Commission.

**LOADS.**—The loads carried by a bridge consist of (1) fixed or dead loads, (2) the moving or live load, and (3) miscellaneous loads.

The dead load consists of the weight of the structure and is always carried by the bridge; the live load consists of the moving load which the bridge is built to carry, while the miscellaneous loads include wind loads, snow loads, etc. Data on dead loads are given in the "Specifications for Steel Highway Bridges" in Appendix I.

**WEIGHTS OF BRIDGES.**—The weight of a bridge is composed of (1) the weight of the steel in the steel framework, consisting of the vertical trusses, the upper and lower lateral systems, the floorbeams, the portals and sway bracing; (2) the weight of the joists and the fence; and (3) the weight of the floor covering.

**WEIGHTS OF STEEL HIGHWAY BRIDGES.**—The following data may be used in calculating the dead loads in the design of highway bridges or as a basis for preliminary estimates.

**AMERICAN BRIDGE COMPANY.—Standard Steel Highway Bridges with Timber Floor.** Timber floor, 3-in. plank on roadway and 2-in. plank on footwalks. Live loads for floor and its supports, 100 lb. per sq. ft. of floor surface, or 6 tons on two axles 10 ft. centers and 5 ft. gage, or a 15-ton road roller. For trusses 100 lb. per sq. ft. of roadway up to a span of 75 ft., 75 lb. per sq. ft. of roadway for spans of 168 ft. and over, and proportional for intermediate spans. No allowance is made for impact. Designed for allowable stresses given in specifications in Appendix I. Let $W$ = weight of the structural steel per lineal foot of span; $L$ = length of span in feet, $b$ = width of roadway in feet (without sidewalks).

1. **Steel Through Plate Girders.**—Through plate girder spans 36 ft. to 70 ft., roadway 20 ft. wide, without sidewalks, but including stringers. The weight of structural steel per lineal foot of span is

$$W = 300 + 3.8L. \tag{1}$$

For sidewalks with steel joists add about 12 lb. per sq. ft. of sidewalks.

2. **Steel Low Riveted Truss Spans, with Timber Floor.**—For low truss spans 36 ft. to 102 ft., with timber floors, the weight of structural steel per lineal foot of span, not including the weight of the stringers and the railing, is given approximately by the formula for a 16-ft. roadway

$$W = 100 + 2.0L. \tag{2}$$

and for a 20-ft. roadway

$$W = 150 + 1.7L. \tag{3}$$

3. **Steel Low Riveted Truss Spans, with Reinforced Concrete Floors.**—For low truss spans 36 ft. to 102 ft., with reinforced concrete floors, 5 in. thick with 6 in. of gravel at center and 3 in. of gravel at curb, the weight of structural steel per lineal foot of span, not including the weight of the stringers and the railing, is given approximately by the formula for a 16-ft. roadway

$$W = 150 + 3.5L. \tag{4}$$

and for a 20-ft. roadway

$$W = 185 + 3.5L. \tag{5}$$

4. **Steel High Truss Spans, with Timber Floor.**—For high truss spans 104 to 204 ft., with timber floors, the weight of structural steel per lineal foot of span, not including the weight of the stringers and the railing, is given approximately by the formula for a 16-ft. roadway

$$W = 250 + 1.5L. \tag{6}$$

and for a 20-ft. roadway

$$W = 285 + 1.2L. \tag{7}$$

**IOWA HIGHWAY COMMISSION.—Steel Highway Bridges with Reinforced Concrete Floor.**—Reinforced concrete floor slabs 6 in. thick for all spans in which stringers are used. Slabs for stringerless floors 7½ in. thick for 8-ft. span, 8 in. thick for 9-ft. span, and 8½ in. thick for 10-ft. span. Live loads for the floor and its supports a uniform live load of 100 lb. per sq. ft., and a 15-ton traction engine with two-thirds of the load on the rear axle; axles spaced 11 ft. centers, and rear wheels spaced 6 ft. centers. Rear wheels 22 in. wide. The trusses are to be designed for the uniform loads given in Table II. No allowance is made for impact.

Let $W$ = weight of structural steel in lb. per lineal foot of span; $L$ = length of span in feet; $b$ = width of span in feet (without sidewalks).

1. **Steel Beam Spans.**—The weight of steel beam spans from 16 ft. to 32 ft. and with 16-ft., 18-ft., and 20-ft. roadway are given in Table I, Chapter XI.

2. **Steel Low Truss Spans, with Stringers.**—For low truss highway bridges with spans of 35 ft. to 85 ft., not including the weight of the fence or the steel stringers, the weight of structural steel per lineal foot of span for a 16-ft. roadway is

$$W = 235 + 2.35L. \tag{8}$$

and for an 18-ft. roadway is

$$W = 240 + 2.40L. \tag{9}$$

3. **Steel Low Truss Spans, without Stringers.**—For low truss highway bridges with spans of 35 ft. to 100 ft., not including the weight of the fence, the weight of the structural steel per lineal foot of span for a 16-ft. roadway is

$$W = 200 + 4L. \tag{10}$$

and for an 18-ft. roadway is

$$W = 225 + 4.25L. \tag{11}$$

**4. Steel High Truss Spans, with Stringers.**—For high through truss highway bridges with spans of from 90 ft. to 150 ft., not including the weight of fence or the steel stringers, the weight of structural steel per lineal foot of span for a 16-ft. roadway is

$$W = 245 + 2.45L. \tag{12}$$

and for an 18-ft. roadway is

$$W = 270 + 2.7L. \tag{13}$$

**WISCONSIN HIGHWAY COMMISSION. Steel highway bridges with reinforced concrete floor.**—Reinforced concrete floor slabs 6 in. thick for all spans. Live loads for the floor and its supports a 15-ton road roller with two-thirds of the load on the rear axle, axles 10 ft. centers, rear rolls 4 ft. 10 in. centers, rear rolls 20 in. wide. The trusses designed for the loads given in Table II. No allowance is made for impact. Let $W$ = weight of structural steel in lb. per lineal foot of span, $L$ = length of span in feet; $b$ = width of roadway in feet (without sidewalks).

**1. Steel Beam Spans.**—Weight of steel beam spans from 10 ft. to 38 ft. and for 16-ft., 18-ft. and 20-ft. roadway are given in Table II, Chapter XI.

**2. Steel Through Plate Girders.**—The weight of the structural steel in through plate girder highway bridges from 35 ft. span to 80 ft. span including floorbeams spaced 3 to 2½ ft. apart, is given approximately by the following formula. For a 16-ft. roadway

$$W = 300 + 3L. \tag{14}$$

For an 18-ft. roadway

$$W = 300 + 3.25L. \tag{15}$$

and for a 20-ft. roadway

$$W = 320 + 4L. \tag{16}$$

**3. Steel Low Truss Spans, with Stringers.**—The weight of the structural steel in low truss steel highway bridges with spans of 35 ft. to 85 ft. span, not including the weight of the fence or the steel stringers, is given approximately by the formula. For a 16-ft. roadway

$$W = 80 + 3.5L. \tag{17}$$

and for an 18-ft. roadway

$$W = 80 + 4L. \tag{18}$$

**4. Steel High Truss Spans, with Stringers.**—For high through truss steel highway bridges with spans of from 90 ft. to 150 ft., not including the weight of the fence or the steel stringers, the weight of structural steel per lineal foot of span is given approximately by the formula. For a 16-ft. roadway

$$W = 180 + 2L. \tag{19}$$

and for an 18-ft. roadway

$$W = 240 + 2L. \tag{20}$$

**ILLINOIS HIGHWAY COMMISSION. Steel highway bridges with reinforced concrete floor.**—Reinforced concrete floor slabs 4 in. thick with a wearing surface assumed to weigh not less than 50 lb. per sq. ft. Live load for floor and its supports a 15-ton traction engine, supported on two axles spaced 10 ft. apart, with two thirds of the load on the rear axle; or a uniform live load of 125 lb. per sq. ft. The trusses designed for the loads given in Table II. No allowance is made for impact.

Let $W$ = weight of steel in lb. per lineal foot of span, $L$ = span of bridge in feet, $b$ = width of roadway in feet (without sidewalks).

**1. Steel Low Truss Spans, with Stringers.**—The weight of the structural steel in low truss steel highway bridges with spans of 50 ft. to 85 ft., not including weight of the fence or the steel stringers, is given approximately by the formula. For a 16-ft. roadway, $b$ = 16 ft.

$$W = 235 + 2.35L. \tag{21}$$

and for an 18-ft. roadway, $b$ = 18 ft.

$$W = 240 + 2.4L. \tag{22}$$

**2. Steel High Truss Spans, with Stringers.**—The weight of structural steel in high truss steel highway bridges with spans of 90 ft. to 160 ft., not including the weight of fence or the steel stringers, is given approximately by the formula. For a 16-ft. span, $b$ = 16 ft.

$$W = 140 + 4L. \tag{23}$$

and for an 18-ft. span, $b$ = 18 ft.

$$W = 180 + 4.5L. \tag{24}$$

The weights given by formulas (21) to (24) are for bridges with concrete floors weighing 100 lb. per sq. ft. Calculations by Mr. Clifford Older, Bridge Engineer, Illinois Highway Commission, show that a variation of the weight of the floor of 10 lb. per sq. ft. makes a similar variation in the weight of the structural steel, including the joists, of 4.35 per cent for a 50-ft. span, of 3.75 per cent for a 160-ft. span, and proportional for intermediate spans. For the structural steel, not including the joists, an average value of 4 per cent may be used for each decrease of 10 lb. per sq. ft. of floor surface.

**BOSTON BRIDGE WORKS STANDARDS.***—The weights of steel highway bridges designed by the Boston Bridge Works are as follows:

Through truss highway bridges without sidewalks designed for a live load of 80 lb. per sq. ft. for the trusses, 100 lb. per sq. ft. and a 6-ton wagon for the floor. The weight, $w$, of steel in lb. per sq. ft. of area covered by the floor, not including joists or fence, for a span of $L$ ft., is

$$w = 5 + L/9.5 \tag{25}$$

The weight of through truss highway bridges with two sidewalks is

$$w = 2.8 + L/11.3 \tag{26}$$

The sidewalks were 5 or 6 ft. wide, and the clear roadways were 16 to 20 ft. The total area covered by the roadway and sidewalk floors is to be used in calculating the weight of steel.

**Weights of Steel Highway Plate Girder Bridges.**—The weights of highway plate girder bridges as designed by the Boston Bridge Works for the live loads shown are as follows.

Deck plate girder highway bridges without sidewalks designed for a live load of 100 lb. per sq. ft. for girders, 100 lb. per sq. ft. and a 6-ton wagon for the floor. The weight, $w$, of steel in lb. per sq. ft. of area covered by the floor, not including joists or fence, for a span of $L$ ft., is

$$w = 2.5 + L/3.4 \tag{27}$$

The weight of deck plate girder highway bridges with sidewalks is

$$w = 2.5 + L/4.4 \tag{28}$$

The weight of through plate girder highway bridges without sidewalks is

$$w = 3 + L/4.25 \tag{29}$$

The weight of through plate girder highway bridges with sidewalks is

$$w = 3.3 + L/5.6 \tag{30}$$

**Weight of Electric Railway Bridges.**—The Boston Bridge Works gives the following formulas for the weight of electric railway bridges, where $W$ = total weight of steel in lb. per lineal foot of bridge and $L$ is the span of the bridge in feet.

Beam bridges

$$W = 50 + 5L \tag{31}$$

* Published by permission of John C. Moses, Chief Engineer.

Light truss bridges

$$W = 200 + 0.8L \tag{32}$$

Heavy truss bridges

$$W = 250 + 1.5L \tag{33}$$

The beam bridges were designed for 30-ton cars; the light truss bridges were designed for 15-ton cars or 1,500 lb. per lineal foot of bridge, and the heavy truss bridges were designed for 30-ton cars, or 2,000 lb. per lineal foot of bridge.

**LIVE LOADS.**—The live loads for highway bridges are usually assumed to consist of a uniform live load for the trusses and a uniform live load or a concentrated moving load for the floor and its supports. A few highway bridge specifications require that trusses be designed for a concentrated moving load as well as for a uniform live load, and also that the floor and its supports be designed for a concentrated moving load and that the portion of the floor of the bridge not covered by the concentrated load be covered with a uniform live load. In calculating the stresses in the truss members the uniform live load is commonly assumed as applied in full joint loads at joints on the loaded chord. Moving loads and loads suddenly applied produce stresses that are greater than the static stresses due to stationary loads or to loads gradually applied. This increase in stress due to moving loads or due to loads suddenly applied is called impact stress.

**IMPACT.**—The effect of impact or increase in live load stresses over the stresses due to the same loads gradually applied, is very much less for highway bridges than for railway bridges. Experiments made by Professor F. O. Dufour and recorded in Journal of Western Society of Engineers, June, 1913, show that the effect of impact on steel truss highway bridges with concrete floors is very small. The effect of impact on steel truss bridges with plank floors is considerably larger than for bridges with concrete floors. The maximum impact percentages do not occur with maximum static stresses. Experiments made at the University of Colorado under the author's direction show that the effect of impact on highway bridges is very much less than for railway bridges.

The specifications of the highway commissions of Illinois, Iowa, Michigan, Nebraska and Wisconsin do not add impact for highway bridges.

The allowance for impact by the Massachusetts Railway Commission is as follows: For stringers, floorbeams and hangers, when loaded with a 20-ton auto truck, 50 per cent; for all other loads, floorbeams and stringers, 25 per cent; floorbeam hangers, 40 per cent; counters, 40 per cent; for all other members in trusses, and for main girders the percentage shall be 26⅔ minus one-twelfth the loaded length in feet, with a maximum of 25 and a minimum of 10 per cent.

Mr. J. A. L. Waddell in "Bridge Engineering" specifies that highway bridges shall be designed for the impact allowance, $I = 100/(nL + 200)$, where $L$ is the loaded length of the bridge in feet that produces maximum stress and $n$ is total clear width in feet of roadway and footwalks divided by twenty. The above impact allowance is made for motor-truck loadings but not for road-roller loadings.

General Specification for Steel Highway Bridges adopted 1918 by the Engineering Institute of Canada specifies impact as follows: Impact shall be added to the maximum computed stresses produced by the specified motor-truck and electric-car loads only. For motor-truck loads, the impact shall be taken as 30 per cent of the statically computed stresses produced thereby. For electric car loads, the impact shall be determined by the formula

$$I = S \cdot 150/(L + 300),$$

in which $I$ = impact stress; $S$ = statically computed maximum stress in member considered due to electric-car loads; $L$ = length of load in feet producing maximum stress in member considered.

The specifications of the West Virginia Highway Commission and the Oregon Highway Commission specify the impact factor, $I = 100/(L + 300)$, where $L$ is the loaded length of the bridge in feet that produces maximum stress in the member.

The Montana Highway Commission specifies 25 per cent impact.

The U. S. Bureau of Public Roads specifies 30 per cent impact.

The Department of Public Roads of Kentucky requires no impact allowance for bridges with concrete floors, and 25 per cent for bridges with wooden floors.

The Utah Highway Commission specifies 25 per cent impact for floors, and 15 per cent for trusses.

For concrete highway bridges the impact allowance varies from no impact allowance, as specified by the highway commissions of Illinois, Iowa, Michigan, Nebraska and Wisconsin; an allowance of 15 per cent of the live load, as specified by the highway commission of West Virginia, to an allowance of 30 per cent of the live load, as specified by the U. S. Bureau of Public Roads. Watson's "General Specifications for Concrete Bridges," third edition, 1916, uses an impact allowance of $I = 150/(L + 300)$, where $L$ is the loaded length of the bridge in feet that produces maximum stress.

**Ketchum's Specifications for Impact.**—The author has adopted the following impact factors for concrete bridges and steel bridges.

(a) For concrete arches with spandrel filling or culverts with a minimum filling of one foot, no allowance for impact.

(b) For concrete slab and girder bridges and trestles, and arches without spandrel filling, 30 per cent for impact.

(c) For steel bridges the following allowance for impact. For the floor and its supports including floor slabs, floor joists, floorbeams and hangers, 30 per cent.

For all truss members other than the floor and its supports, the impact increment shall be $I = 100/(L + 300)$, where $L$ = length of span for simple highway spans (for trestle bents, towers, movable bridges, arch and cantilever bridges, and for bridges carrying electric trains, $L$ shall be taken as the loaded length of the bridge in feet producing maximum stress in the member).

**CONCENTRATED LIVE LOADS.**—Traction engines weighing 20 tons are quite common in the west and northwest. The heaviest motor truck in common use has a capacity of $7\frac{1}{2}$ tons and a total weight of 13 tons, with nearly 10 tons on the rear axle. With an overload of 50 per cent, which is not unusual, this truck would carry 14 tons on the rear axle. The maximum road roller weighs 20 tons.

The highway commissions of the different states have adopted concentrated live loads as follows: Illinois specifies a 15-ton traction engine; Iowa specifies a 15-ton traction engine for bridges with reinforced concrete floors; Wisconsin specifies a 15-ton road roller; Michigan specifies an 18-ton road roller; Nebraska specifies a 20-ton traction engine; Minnesota specifies a 20-ton traction engine; New York specifies a 15-ton road roller; all loadings to be used without impact.

Utah specifies an 18-ton road roller with 25 per cent impact; Oregon specifies a 15-ton road roller for medium traffic and a 20-ton road roller for heavy traffic; Montana specifies a 20-ton traction engine with 25 per cent impact; the Massachusetts Railway Commission specifies a 20-ton motor truck with an allowance of 50 per cent for impact on the floor and its supports; Mr. J. A. L. Waddell in "Bridge Engineering" specifies for class A bridges an 18-ton motor truck with impact allowance as given above.

The Ohio State Highway Commission specifies a 20-ton concentrated load on two axles spaced 10 ft., wheels with gage of 6 ft. with two-thirds on rear axle on roads in industrial communities, and 15 tons with same spacing and distribution on country highways. Impact for bridges with concrete floors is one-half that given for steel bridges in Appendix I.

The U. S. Bureau of Public Roads specifies a 15-ton truck for all types of highway bridges except timber bridges for which a 10-ton truck is specified. Each truck has axles 10 ft. centers, and wheels 6 ft. centers, with two-thirds of the total load carried on the rear axle. All loads with 30 per cent impact.

General Specification for Steel Highway Bridges adopted 1918 by the Engineering Institute of Canada specifies motor-truck loadings as follows: For city bridges a 25-ton motor-truck with

axles spaced 12 ft. and wheels with an 8-ft. gage. Two-thirds of load on rear axle. Rear wheels 34 in. wide. For bridges in towns and country highways a 15-ton motor-truck with axles spaced 10 ft. and wheels with an 8-ft. gage. Two-thirds of load on rear axle. Rear wheels 18 in. wide. For bridges in remote mountainous highways an 8-ton motor truck with axles spaced 8 ft. and wheels with an 8-ft. gage. One-half of load on rear axle. Rear wheels 6 in. wide.

For additional data see article entitled "Concentrated Live Loads for Highway Bridges," by Milo S. Ketchum, printed in University of Colorado Journal of Engineering, October, 1916.

**Ketchum's Specifications for Concentrated Moving Loads.**—The author has adopted the following specifications for concentrated moving loads.

(a) That highway bridges on main roads or near towns or cities shall be designed to carry a 20-ton motor truck with axles spaced 12 ft. and wheels 6-ft. centers on axle, with 14 tons on rear axle and 6 tons on front axle. The truck to occupy a space 10 ft. wide and 32 ft. long. The rear wheels to have a width in inches equal to the total load in tons (20 in. for a 20-ton truck).

(b) That bridges not on main roads shall be designed for a 15-ton motor truck with axles spaced 10 ft. and wheels 6-ft. centers on axle, and occupying a space 10 ft. wide and 30 ft. long, with 10 tons on rear axle and 5 tons on front axle, and with rear wheels 15 in. wide.

(c) To provide for impact and vibration and unevenness of road surface thirty (30) per cent is to be added to the maximum live load stresses. Only one motor truck is to be assumed to be on a bridge at one time.

Motor trucks have narrower tires and are driven at greater speeds than traction engines, and therefore not only produce greater static stresses in the floor, but should have a greater impact allowance. In view of the above, it would not appear to be necessary to consider any road rollers or traction engines now in use in addition to the above motor-truck loadings.

**DISTRIBUTION OF CONCENTRATED LOADS.**—In designing floor slabs, floor stringers and floorbeams it is necessary to know the distribution of the concentrated loads.

**Concrete Floor Slabs.**—Tests of the distribution of concentrated loads on concrete floor slabs have been made by the Ohio Highway Commission, the results of which are given in Bulletin No. 28, published by the Commission; by Mr. W. A. Slater at the University of Illinois and described in Proceedings of American Society for Testing Materials, Vol. XIII, 1913, and by A. T. Goldbeck and E. B. Smith, described in Journal of Agricultural Research, Vol. VI, No. 6, Department of Agriculture, Washington, D. C., May 8, 1916.

*Ohio Tests.*—The following conclusions drawn from the Ohio tests are of interest:

"The percentage of reinforcement has little or no effect upon the distribution to the joists, so long as safe loads on the slabs are not exceeded.

"The outside joists should be designed for the same total live load as the intermediate joists.

"The axle load of a truck may be considered as distributed over 12 ft. in width of roadway.

"The safe value for 'effective width' of a slab, where the total width of slab is greater than $1.33\,L + 4$ ft. is given by the formula, $e = 0.6L + 1.7$ ft., where $e$ = effective width (width over which a single concentrated load may be considered as uniformly distributed on a line down the middle of the slab parallel to the supports) and $L$ = span in feet."

*Slater Tests.*—It was recommended that where the total width of slab is greater than twice the span, the effective width be taken as $e = 4x/3 + d$, where $x$ is the distance from the concentrated load to the nearest support, and $d$ is the width at right angles to the support over which the load is applied. While the depth of slab and the amount of longitudinal reinforcement had little effect on the distribution, it was recommended that the latter be limited to 1 per cent.

*Goldbeck and Smith Tests.*—Tests were made on three slabs, each slab being 32 ft. wide, 16 ft. span, and with effective depths of 10.5 in., 8.5 in. and 6 in., respectively. All slabs were made of 1–2–4 Portland cement concrete, and were reinforced with 0.75 per cent of mild steel.

The following conclusions were drawn from these tests:

(1) The effective width decreases as the effective depth increases; the effective width for safe loads being 75.7 per cent; 81.1 per cent, and 109.3 per cent of the span, for the slabs having effective depths of 10.5 in., 8.5 in. and 6 in., respectively.

(2) For slabs in which the ratio of the width of the slab is not less than twice the span length, the effective width may be taken as

$$e = 0.7L \tag{34}$$

where $e$ is the effective width and $L$ is the span length.

(Additional tests by Goldbeck, Proceedings American Concrete Institute, 1917, show that formula (34) may be used when the width of the slab is not less than the span.)

Watson's "General Specifications for Concrete Bridges," third edition, 1916, specifies that concentrated loads on reinforced concrete slabs may be assumed as distributed over a distance of 4 ft. at right angles to the supports, and a distance parallel to the supports equal to 2 ft. plus three-tenths of the span of the slab.

The State Highway Department of Ohio uses the following distribution of concentrated loads on floor slabs.

For spans less than 6 ft. the percentage, $p$, of the wheel load carried by one foot in width of slab for a span in feet, $l$, is given by the formula

$$p = 42 - 4l \tag{35}$$

while for spans greater than 6 ft. the percentage, $p'$, of the wheel load carried by one foot in width of slab for a span in feet, $l$, is given by the formula

$$p' = 20 - 0.4l \tag{36}$$

For a span of 5½ ft., from formula (35), $p = 20$ per cent, and the concentrated load is assumed as carried by a slab 5 ft. wide, applied on a line parallel to the supports.

For a span of 10 ft., from formula (36), $p' = 16$ per cent, and the concentrated load is assumed as carried by a slab 6.67 ft. wide, applied on a line parallel to the supports.

The U. S. Bureau of Public Roads specifies that wheel loads be distributed on concrete slabs as follows: with a fill of ballast or paving of 8 in. or less, 9 to 17 in., inclusive, and 18 in. or more, an area of distribution of 4 ft. square, 5 ft. square, and 6 ft. square, respectively.

**Plank Floor on Steel Stringers.**—A series of experiments to determine the distribution of concentrated loads on a timber floor supported on steel stringers has been made at Iowa State College of Agriculture and Mechanic Arts by T. R. Agg and C. S. Nichols, and published in Engineering Experiment Station Bulletin 53. The floor consisted of a 3-in. plank floor supported on 6-in. I-beams spaced 12 in. centers and also spaced 19 in. centers; and a 3-in. plank floor supported on 7-in. I-beams spaced 24 in. centers and also spaced 27 in. centers. The concentrated loads were applied through wheels 6′ 8″ in diameter and 24 in. wide. The wheels were spaced 6 ft. centers. A summary of the tests shows:

(1) For stringers spaced 12 in. centers the maximum load carried by a single stringer was 25 per cent of a wheel load.

(2) For stringers spaced from 24 in. to 27 in. centers the maximum load carried by one stringer was 55 per cent of a wheel load. A top floor of 2-in. plank laid longitudinally reduced the concentration under a wheel slightly.

(3) The concentration on stringers immediately under the wheels was slightly increased when the ends of the floor planks were not bolted down.

(4) The concentration on the outer stringer increases rapidly as the load approaches the side, and the outer stringer should have the same section modulus as the intermediate stringers.

These tests check the rule that the percentage of a concentrated load carried by one stringer is equal to the stringer spacing in feet divided by four feet.

**Floor Stringers and Floorbeams.**—The Illinois Highway Commission specifies that longitudinal stringers be spaced not more than 2½-ft. centers, and that each stringer be designed for 20

per cent of the rear axle load concentrated at the center of the span when a concrete sub-floor is used, and 25 per cent of the rear axle load when a plank floor is used. Transverse stringers or floorbeams, spaced not more than $2\frac{1}{2}$-ft. centers, shall be designed to carry 40 per cent of the rear axle load distributed over the middle 10 ft. of the stringer. Floorbeams shall be designed for maximum stresses due to concentrated load.

The Iowa Highway Commission specifies that one-third of a wheel load be assumed as carried by one joist, when a concrete floor slab is used, and that one-half of a wheel load be assumed as carried by one joist, when a plank floor is used.

The Massachusetts Railway Commission specifies that the wheel load on plank floors be distributed over a width in feet equal to the thickness of the floor in inches, with a maximum distribution of 6 ft. With solid floors each wheel load is assumed as distributed over a width of 6 ft.

Watson's "General Specifications for Concrete Bridges," third edition, 1916, specifies that the part of the concentrated load carried by one stringer shall be found by dividing the stringer spacing by the gage distance of the concentrated load. With a gage distance of 6 ft. this gives one-third the total load for a stringer spacing of 2 ft.; one-half the total load for a stringer spacing of 3 ft.; the total load for a stringer spacing of 6 ft.

General Specification for Steel Highway Bridges adopted 1918 by the Engineering Institute of Canada specifies that roadway stringers or joists shall be designed to carry proportions of the motor truck loads as given by the formula $C = P \cdot d/g$, where $C$ = proportion of front or rear wheel-load supported by one stringer; $P$ = concentration on one wheel, front or rear; $d$ = distance center to center of stringers; $g$ = gage, center to center of wheels.

The U. S. Bureau of Public Roads specifies that loads be distributed on stringers and floorbeams as follows: For bridges with a timber floor and longitudinal stringers, stringers spaced 2 ft. centers shall be assumed as carrying one-half the wheel load, stringers spaced 3 ft. centers shall be assumed to carry three-fourths the wheel load, and proportional for other spacings. For bridges with concrete floor on steel or reinforced concrete longitudinal stringers, stringers spaced 4 ft. centers shall be assumed to carry two-thirds of the full load, stringers spaced 6 ft. centers shall be assumed to carry the full load, and proportional for other spacings. Outside stringers shall be placed inside the curb and shall have at least as much strength as the interior stringers. For bridges with concrete floor carried on steel floorbeams without stringers, each floorbeam shall be assumed to carry the full load for spacings of 5 ft. to 10 ft. For spacings of floorbeams less than 5 ft. the fraction of the load carried by one beam shall be equal to the spacing of the floorbeams in feet divided by 5 ft. The wheel load shall be assumed as uniformly distributed along the floorbeam for depths of ballast or paving of 8 in. or less, 9 in. to 17 in., and 18 in. or more, a distance of 9 ft., 10 ft. and 11 ft., respectively.

**Ketchum's Specifications for Distribution of Concentrated Loads.**—From a study of the various tests and specifications, the author has adopted the following rules for calculating the stresses in slabs, stringers and floorbeams:

(*a*) The distribution of concentrated wheel loads for bending moments in reinforced concrete slabs with longitudinal girders shall be calculated by the formula

$$e = \tfrac{2}{3}(l + c) \tag{37}$$

with a maximum limit of 6 ft. for $e$, where $e$ = effective width (distance that the load may be considered as uniformly distributed on a line down the middle of the slab parallel to the supports), $l$ = span, and $c$ = width of tire of wheel, all distances in feet. See Fig. 1.

(*b*) The distribution of concentrated wheel loads for bending moments in reinforced concrete slabs with transverse girders shall be calculated by the formula

$$e = 2l/3 + c \tag{38}$$

with a maximum limit of 6 ft. for $e$, where $e$ = effective width, $l$ = span, and $c$ = width of tire of wheel as defined in paragraph (a). See Fig. 2.

(*c*) The distribution of concentrated wheel loads for bending moments in slabs of girder bridges in which the span of the bridge is not less than the width of bridge center to center of girders, shall be calculated for spans of 9 ft. or over by the formula

$$e = 2l/3 \tag{39}$$

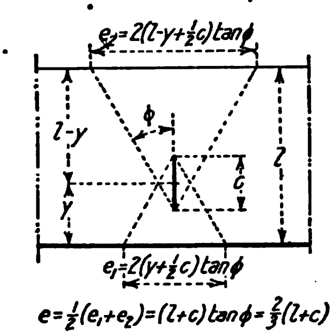



FIG. 1.

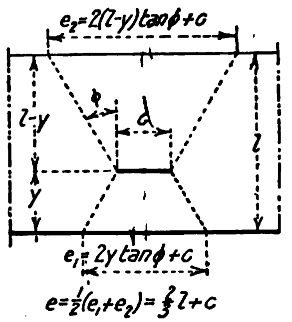



FIG. 2.

with a maximum limit of $e$ = 12 ft., where $e$ = effective width, and $l$ = span as defined in paragraph (a).

## TABLE I.

### Distribution of Concentrated Loads on Slabs.

*Effective Width of Slab for Concentrated Load Distributed on a Line.*

| Spacing of Stringer, Ft. | 20-Ton Auto Truck, Wheel 20 In. Wide. | | 15-Ton Auto Truck, Wheel 15 In. Wide. | | 12-Ton Auto Truck, Wheel 12 In. Wide. | | 10-Ton Auto Truck, Wheel 10 In. Wide. | |
|---|---|---|---|---|---|---|---|---|
| | Effective Width. | | Effective Width. | | Effective Width. | | Effective Width. | |
| | Longitudinal Stringer. | Transverse Stringer. | Longitudinal Stringer. | Transverse Stringer. | Longitudinal Stringer. | Transverse Stringer. | Longitudinal Stringer. | Transverse Stringer. |
| | Ft. | Ft. | Ft. | Ft. | Ft. | Ft. | Ft. | Ft. |
| 2 | 2.4 | 3.0 | 2.2 | 2.6 | 2.0 | 2.2 | 1.9 | 2.0 |
| 3 | 3.1 | 3.7 | 2.8 | 3.3 | 2.7 | 2.8 | 2.6 | 2.6 |
| 4 | 3.8 | 4.3 | 3.5 | 3.9 | 3.3 | 3.5 | 3.2 | 3.3 |
| 5 | 4.4 | 5.0 | 4.2 | 4.6 | 4.0 | 4.2 | 3.9 | 4.0 |
| 6 | 5.1 | 5.7 | 4.8 | 5.3 | 4.7 | 4.8 | 4.6 | 4.6 |
| 7 | 5.8 | 6.0 | 5.5 | 5.9 | 5.4 | 5.5 | 5.3 | 5.3 |
| 8 | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 | 6.0 |

Effective width on line along center of beam for moment, and near end of beam for shear. Minimum effective width of shear is 3 ft.

(*d*) The effective width for shear in beams carrying concentrated loads shall be taken the same as for bending moment as calculated by formula (37) or formula (38), with a minimum effective width of 3 ft. and a maximum effective width of 6 ft.

The total shear for an effective width of 3 ft. shall be considered as punching (pure) shear. The total shear for an effective width of 4.5 ft. and over shall be considered as beam shear (a measure of diagonal tension), for effective widths between 3 ft. and 4.5 ft. the total shear shall be divided proportionally between punching shear and beam shear. Beam shear shall be used in calculating bond stress and as a measure of diagonal tension.

(*e*) In the design of longitudinal joists or stringers with concrete floors, the fraction of the concentrated load carried by one stringer for spacings 6 ft. or less shall be taken equal to the stringer spacing in feet divided by 6 ft.; with plank floors the fraction of the concentrated load carried by one stringer for spacings 4 ft. or less shall be taken equal to the stringer spacing in feet divided by 4 ft., the maximum in each case being the full load. Outside stringers shall be designed for the same load as intermediate stringers.

(*f*) In the design of transverse stringers or floorbeams with concrete floors, the fraction of the concentrated load carried by one floorbeam for floorbeams spaced 6 ft. or less, shall be taken equal to the floorbeam spacing divided by 6 ft. For floorbeams spaced 6 ft. or over the entire reactions are assumed as carried by one floorbeam. Axle loads are assumed as distributed on a line 12 ft. long.

The distribution of concentrated loads calculated for different auto trucks for formulas (37) and (38) are given in Table I.

**UNIFORM LIVE LOADS FOR TRUSSES.**—The uniform live loads for trusses of steel highway bridges as specified by the highway commissions of Illinois, Iowa and Wisconsin, the American Concrete Institute, 1916, and the uniform loads as specified by the author for classes $D_1$ and $D_2$ are given in Table II. The $D_1$ and $D_2$ loadings are to be taken as proportional for intermediate spans, and are to be increased for impact.

It will be seen that the $D_1$ loadings with impact added are practically the same as the Illinois loadings; while the $D_2$ loadings with impact added are practically the same as the Iowa and Wisconsin loadings.

TABLE II.

UNIFORM LIVE LOADS FOR HIGHWAY BRIDGES.

| Illinois Highway Commission. | | Iowa Highway Commission. | | Wisconsin Highway Commission. | | American Concrete Institute, 1916. | | | | Ketchum's Specifications, 1918. | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Class A. | | Class B. | | Class $D_1$. | | Class $D_2$. | |
| Span, Ft. | Load, Lb./Sq. Ft. | Span, Ft. | Load, Lb./Sq. Ft. | Span, Ft. | Load, Lb./Sq. Ft. | Span, Ft. | Load, Lb./Sq. Ft. | Span, Ft. | Load, Lb./Sq. Ft. | Span, Ft. | Load, Lb./Sq. Ft. | Span, Ft. | Load, Lb./Sq. Ft. |
| Up to 50 | 125 | Up to 50 | 100 | Up to 40 | 125 | Up to 80 | 125 | Up to 80 | 100 | 30 | 125 | 30 | 100 |
| 50–100 | 100 | 50–100 | 90 | 50 | 120 | 80–100 | 110 | 80–100 | 90 | 50 | 106 | 50 | 90 |
| 100–150 | 100 | 100–150 | 80 | 75 | 106 | 100–125 | 100 | 100–125 | 80 | 80 | 85 | 80 | 75 |
| 150–200 | 85 | 150–200 | 70 | 100 | 93 | 125–150 | 90 | 125–150 | 75 | 100 | 80 | 100 | 71 |
| Over 200 | 85 | 200–250 | 50 | 150 | 60 | 150–200 | 85 | 150–200 | 65 | 160 | 68 | 160 | 60 |
| | | Over 250 | 50 | 180 and over | 50 | Over 200 | 80 | Over 200 | 60 | 200 and over | 60 | 200 and over | 50 |
| Class $D_1$ and $D_2$ bridge loadings to be increased for impact. | | | | | | | | | | | | | |

General Specification for Steel Highway Bridges adopted 1918 by the Engineering Institute of Canada specifies uniform live loads for trusses as follows:

Class A, city bridges, 100 lb. per sq. ft. for spans of 100 ft. or less; 80 lb. per sq. ft. for spans of 200 ft. and over, and proportional for spans between 100 ft. and 200 ft. Minimum load per lineal foot, 1,200 lb.

Class B, town and country bridges, 80 lb. per sq. ft. for spans of 100 ft. or less, 60 lb. per sq. ft. for spans of 200 ft. and over, and proportional for spans between 100 ft. and 200 ft. Minimum load per lineal foot, 900 lb.

Class C, remote highway bridges, 70 lb. for spans of 50 ft. or less, 40 lb. for spans of 200 ft. and over, and proportional for spans between 50 ft. and 200 ft. Minimum load per lineal foot, 600 lb.

All of above loadings are used without impact.

**UNIFORM LIVE LOADS FOR FLOORS.**—The Illinois Highway Commission specifies that stringers and floorbeams for spans of 50 ft. and less shall be designed for a uniform live load of 125 lb. per sq. ft., and spans over 50 ft. in length for a uniform live load of 100 lb. per sq. ft., or a 15-ton concentrated load for all spans. No allowance is made for impact.

The Iowa Highway Commission specifies a live load of 100 lb. per sq. ft. or a 15-ton traction engine for class "A" floors, and a live load of 100 lb. per sq. ft., or a 10-ton traction engine for class "B" floors (plank floors). No allowance is made for impact.

The Wisconsin Highway Commission specifies that floor systems and spans under 40 ft. be designed for a 15-ton road roller. No allowance is made for impact.

The Michigan Highway Commission specifies that the floor and its supports be designed for an 18-ton road roller, or 100 lb. per sq. ft. No allowance is made for impact.

The floor systems for $D_1$ bridges are to be designed for 125 lb. per sq. ft. or a 20-ton auto truck; while $D_2$ bridges are to be designed for 100 lb. per sq. ft. or a 15-ton auto truck. An impact factor of 30 per cent is to be added both for the uniform loads and for the auto truck.

**WIND LOADS FOR HIGHWAY BRIDGES.**—The Illinois Highway Commission specifies a wind load of 25 lb. per sq. ft. on the vertical projection of both trusses and the floor system, but in no case shall the wind be less than 300 lb. per lineal foot on the loaded chord nor less than 150 lb. per lineal foot on the unloaded chord.

The Iowa Highway Commission specifies 150 lb. per lineal foot on the unloaded chord and 300 lb. per lineal foot on loaded chord, all loads considered as moving loads.

The Wisconsin Highway Commission specifies 150 lb. per lineal foot on the unloaded chord and 300 lb. per lineal foot on the loaded chord; 150 lb. of the latter being considered a moving load.

Cooper's 1909 specifications require that highway bridges be designed for a lateral force of 150 lb. per lineal foot on the unloaded chord, and a lateral force of 300 lb. per lineal foot on the loaded chord, 150 lb. of the load on the loaded chord being treated as a moving load. For spans exceeding 300 ft. add in each case above 10 lb. for each additional 30 ft.

General Specification for Steel Highway Bridges adopted 1918 by the Engineering Institute of Canada specifies wind loads as follows: A wind load of 300 lb. per lineal foot on loaded chord, and 150 lb. per lineal foot on unloaded chord, both to be treated as moving loads. Viaduct towers are to be designed for a wind force of 50 lb. per sq. ft. on one and one-half times the vertical projection of the structure, unloaded; or 30 lb. per sq. ft. on same surface, plus 150 lb. per lineal foot, applied 5 ft. above floor, when structure is loaded. The longitudnal bracing of towers shall be proportioned for same loads as the transverse bracing.

The author's specifications for wind loads are given in "General Specifications for Steel Highway Bridges" given in Appendix I.

**SNOW LOAD.**—Snow load is usually not considered separately. In localities where the snow is heavy the snow load should be taken into account. Loose and packed snow may be assumed to weigh 5 and 12 lb. per cubic foot, respectively.

**LIVE LOADS FOR ELECTRIC RAILWAY BRIDGES.**—The live loads specified by Mr. C. C. Schneider, M. Am. Soc. C. E., in his Specifications for Electric Railway Bridges are as follows:

§ 11. **Moving Load.**—The moving load shall consist of one of the following classes:

*Class A.*—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are loaded with a load of 40,000 lb., making a total load of 160,000 lb. Or a uniform load of 6,000 lb. per lineal foot for all spans up to 50 ft., reduced to 4,500 lb. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

*Class B.*—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are loaded with a load of 25,000

lb., making a total load of 100,000 lb. Or a uniform load of 3,500 lb. per lineal foot for all spans up to 50 ft., reduced to 2,000 lb. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

*Class C.*—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are loaded with a load of 20,000 lb., making a total load of 80,000 lb. Or a uniform load of 2,500 lb. per lineal foot for all spans up to 50 ft., reduced to 1,500 lb. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

General Specification for Steel Highway Bridges adopted 1918 by the Engineering Institute of Canada specifies three electric car loads as follows:

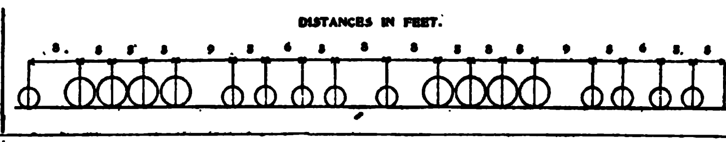
*Class A.*—On each track two electric cars weighing 90,000 lb. each; each car on two pairs of axles, axles of pairs being spaced 5 ft. centers, while the distance of centers of pairs is 25 ft., the pairs of trucks being spaced 20 ft. centers. The distance center to center of axles in adjacent electric cars is 10 ft. Total length of two cars is 90 ft.

*Class B.*—On each track two electric cars weighing 80,000 lb. each, with same dimensions as cars in Class A.

*Class C.*—Two cars weighing 30,000 lb. each, each car on two axles spaced 7 ft. centers, with 14 ft. centers of axles in adjacent cars. Total length of two cars is 56 ft.

**LIVE LOADS FOR RAILWAY BRIDGES.**—The live loads on railway bridges are properly a series of moving concentrated loads. The loads may be considered to consist of a series of wheel loads due to one or more locomotives, followed by a uniform train load; or as an equivalent uniform load.

**Concentrated Loads.**—The most common wheel concentration loading is Cooper's Conventional System, in which two consolidation locomotives are followed by a uniform train load. The spacings for the wheels of all loadings are constant, the loads on the wheels being propor-




| Class | | | | | | | | | | | | | | | | | | | Uniform Load. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| E 50 | 25000 | 50000 | 50000 | 50000 | 50000 | 32500 | 32500 | 32500 | 32500 | 25000 | 50000 | 50000 | 50000 | 50000 | 32500 | 32500 | 32500 | 32500 | 5000 lbs. per lin ft. |
| E 45 | 22500 | 45000 | 45000 | 45000 | 45000 | 29250 | 29250 | 29250 | 29250 | 22500 | 45000 | 45000 | 45000 | 45000 | 29250 | 29250 | 29250 | 29250 | 4500 lbs. per lin. ft. |
| E 40 | 20000 | 40000 | 40000 | 40000 | 40000 | 26000 | 26000 | 26000 | 26000 | 20000 | 40000 | 40000 | 40000 | 40000 | 26000 | 26000 | 26000 | 26000 | 4000 lbs. per lin. ft. |
| E 30 | 15000 | 30000 | 30000 | 30000 | 30000 | 19500 | 19500 | 19500 | 19500 | 15000 | 30000 | 30000 | 30000 | 30000 | 19500 | 19500 | 19500 | 19500 | 3000 lbs. per lin. ft. |

FIG. 3. COOPER'S LOADINGS.

tional in each case. Cooper's loadings are shown in Fig. 3. It will be seen that Cooper's E 50 loading has the same wheel spacings as E 40, all loads being $\frac{5}{4}$ of the loads for E 40.

In bridges designed for Class E 40 loading and under the floor system must in addition be designed for two moving loads of 100,000 lb. each, spaced 6′ 0″ apart on each track. The corresponding loads for Class E 50 are 120,000 lb. with the same spacing. The American Railway Engineering Association has adopted Cooper's loadings, except that the special loads are spaced 7′ 0″. A moment table for Cooper's E 60 loading is given in Table II, Chapter IV.

**Equivalent Uniform Loads.**—An equivalent uniform load is one which approximately produces the same stresses as are produced by a series of concentrated loads. The equivalent uniform load for bending moment in a bridge is the uniform load that will produce the same bending moment at the quarter point of a bridge as the actual wheel loads. The equivalent uniform loads for bridge trusses up to 300 ft. span for Cooper's E 40 loading are given in Fig. 4. For a discussion of the stresses in railway bridge trusses, see Chapter IV.

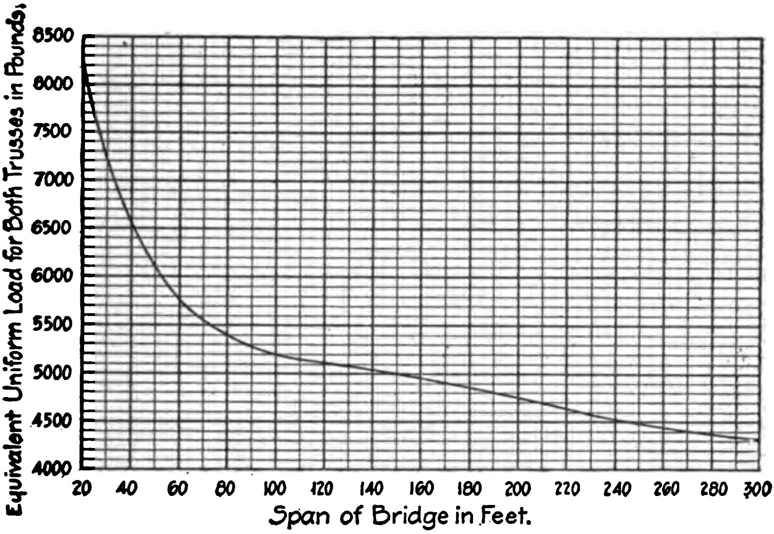



FIG. 4. EQUIVALENT UNIFORM LIVE LOAD FOR COOPER'S E 40 LOADING FOR RAILWAY BRIDGES.

# CHAPTER X.

## DESIGN OF HIGHWAY BRIDGE FLOORS.

**TYPES OF FLOORS.**—The choice of floor for a highway bridge depends upon the traffic, the cost including first cost and cost of maintenance, and the climate. A highway bridge floor consists of a sub-floor which has the necessary strength to carry the loads, and a wearing surface. Plank floors and reinforced concrete slabs without wearing surface have the sub-floor and wearing surface combined. A highway bridge floor should have a strength and a weight appropriate to the structure of the bridge, and should be well drained. The wearing surface should be waterproof, capable of resisting wear and should be as smooth as possible without being slippery. For proper drainage the wearing surface should have a longitudinal grade of not less than 1 in 100 or a transverse slope of not less than 1 in. in 12 ft. The Ohio Highway Commission requires "Water-tight floors on bridges shall, if on a grade of less than ½ per cent be crowned at least one inch for each 20 ft. in width." Sub-floors for highway bridges are made (1) of reinforced concrete; (2) of buckle plates, and (3) of timber. The most common wearing surfaces for highway bridge floors are (*a*) concrete, (*b*) bituminous concrete, (*c*) asphalt, (*d*) creosoted timber blocks, (*e*) brick, (*f*) stone block, (*g*) macadam, (*h*) gravel or earth. The different types of sub-floors and wearing surfaces for highway bridges will be described in some detail.

**REINFORCED CONCRETE FLOOR SLABS.**—Reinforced concrete floor slabs on steel highway bridges may be supported on joists or stringers and floorbeams, or by the floorbeams alone. Stringers are used for beam bridges and are commonly used for truss bridges, while the stringerless floor is commonly used on plate girder bridges. The sub-floor slabs are commonly calculated to carry the dead load due to the weight of the slab and of the wearing surface, and a live load consisting of a uniform load per square foot or a concentrated moving load. The thickness of reinforced concrete slabs in short spans is commonly determined by the concentrated moving load. The stresses in reinforced concrete slabs due to a concentrated load will depend upon the distribution of the load over the slab. The different methods for the distribution of concentrated loads in use in different specifications are described and the specifications adopted by the author are given in Chapter IX. The distribution adopted by the author is given in Table I, Chapter IX.

**Design of Reinforced Concrete Floor Slabs.**—The author's specifications for the design of reinforced concrete floor slabs are as follows:

(1) The floor for all classes of highway bridges except $D_2$ shall be designed for the actual dead load and a uniform live load of 125 lb. per sq. ft. or a 20-ton auto truck, with axles 12 ft. centers and wheels 6 ft. centers, 14 tons to be carried on rear axle and 6 tons on front axle. The rear wheels to have a width of 20 inches. The floors for $D_2$ are to be designed for the actual dead load and a uniform live load of 100 lb. per sq. ft. or a 15-ton auto truck, with axles 10 ft. centers and wheels 6 ft. centers, 10 tons to be carried on the rear axle and 5 tons on the front axle. The rear wheels to have a width of 15 inches. To provide for impact 30 per cent shall be added to all live load stresses in floor slabs.

(2) The distribution of concentrated wheel loads for moments in slabs carried on longitudinal girders shall be as given in Table I, Chapter IX.

(3) The distribution of concentrated wheel loads for moments in slabs carried by transverse floorbeams alone shall be as given in Table I, Chapter IX.

(4) The concrete slab shall be designed for the following allowable unit stresses; the concrete to be made of one part Portland cement, 2 parts clean, sharp sand, and 4 parts suitable gravel or

broken stone that will pass a 1½ in. ring, and that will give a compressive strength of not less than 2,000 lb. per sq. in. when tested in cylinders 8 in. in diameter and 16 in. long after having been stored for 28 days in a moist closet. Allowable compression in slabs, 650 lb. per sq. in., allowable tensile stress in steel, 16,000 lb. per sq. in., modulus of elasticity of steel to be taken as 15 times the modulus of elasticity of concrete, allowable shear as a measure of diagonal tension 40 lb. per sq. in., punching shear 120 lb. per sq. in., bond stress in slabs 80 lb. per sq. in. for plain bars.

(5) The distribution of concentrated loads for shear shall have a minimum effective length of 3 ft. For an effective length of 3 ft. the allowable average shearing stress shall be taken at 120 lb. per sq. in.; for an effective length of 4½ ft. and over the allowable average shearing stress shall be taken at 40 lb. per sq. in., and proportional values between an effective width of 3 ft. and 4½ ft. The concentrated load in each case to be placed so as to produce maximum end shear.

The depths of reinforced concrete slabs required to carry 20-ton, 15-ton, 12-ton and 10-ton auto trucks with an allowance of 30 per cent for impact, when supported on longitudinal joists or stringers are given in Table I, and the thickness of floor slabs when supported on cross floorbeams (stringerless floor) are given in Table II. The reinforcing steel for reinforced concrete floor slabs is given in Table III. The reinforcement given in the table is to be placed at the bottom of slabs calculated as simply supported and at top and bottom of slabs calculated as continuous or partially continuous.

## TABLE I.

### THICKNESS OF REINFORCED CONCRETE FLOOR SLABS, USED WITH JOISTS.

| Simply Supported, Reinforcement on Under Side Only. | | | | | | | Fully Continuous, Reinforcement on Both Sides. | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Span, Ft. | 12-Ton Truck. | | 15-Ton Truck. | | 20-Ton Truck. | | Span, Ft. | 12-Ton Truck. | | 15-Ton Truck. | | 20-Ton Truck. | |
| | Weight of Wearing Surface, Lb. per Sq. Ft. | | | | | | | Weight of Wearing Surface, Lb. per Sq. Ft. | | | | | |
| | 0 | 100 | 0 | 100 | 0 | 100 | | 0 | 100 | 0 | 100 | 0 | 100 |
| | in. | in. | in. | in. | in. | in. | | in. | in. | in. | in. | in. | in. |
| 2 | 5¼ | 5¼ | 5½ | 5½ | 5½ | 5½ | 2 | 4½ | 4½ | 4¾ | 4¾ | 4¾ | 4¾ |
| 3 | 5¾ | 6 | 6¼ | 6¼ | 6½ | 6¾ | 3 | 5 | 5 | 5¼ | 5¼ | 5½ | 5½ |
| 4 | 6¼ | 6½ | 6½ | 6¾ | 7 | 7½ | 4 | 5¼ | 5¼ | 5½ | 5¾ | 6 | 6¼ |
| 5 | 6½ | 6¾ | 6¾ | 7 | 7¾ | 8 | 5 | 5½ | 5¾ | 5¾ | 6 | 6½ | 6½ |
| 6 | 6¾ | 7 | 7½ | 7¾ | 8¼ | 8½ | 6 | 5¾ | 6 | 6 | 6¼ | 6¾ | 7 |

Center of reinforcing 1 in. from face of slab. Impact 30 per cent.
Reinforced as in Table III.

**Examples of Reinforced Concrete Floors.**—Reinforced concrete floor slabs used by the Wisconsin Highway Commission, the Iowa Highway Commission, and the Michigan State Highway Department are given in Chapters XI to XIII. Reinforced concrete floors used by the Ohio State Highway Commission are given in Table III, Chapter XXI.

The Illinois Highway Commission (1917) for stringer spacings of 2½ ft. uses a concrete sub-floor 4 in. thick, with a 4 in. concrete wearing surface, or a 3 in. creosoted timber block wearing surface. The concrete sub-floor, 4 in. thick, is reinforced on the under side with ½ in. square bars, spaced 6 in. centers and centers 1 in. above lower edge. Transverse reinforcement consists of ⅜ in. square bars spaced 12 in. centers. The concrete is specified as 1–2–3½ mix, and is designed for a stress of 800 lb. per sq. in. Details of the standard highway bridge floors of the Illinois Highway Commission are shown in Fig. 2.

(The present (1919) practice of the Illinois Highway Commission is to use 6-in. reinforced concrete floor slabs on steel highway bridges. No covering is used, the upper two inches being considered as a wearing surface.)

The West Virginia Highway Commission specifies 1–2–4 concrete and a minimum thickness of slab of 5 in. to the center of the tension reinforcement.

## TABLE II.

### THICKNESS OF REINFORCED CONCRETE FLOOR SLABS, USED WITHOUT JOISTS.

| Simply Supported, Reinforcement on Under Side Only. | | | | | | | Partially Continuous, Reinforcement on Both Sides. | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Span, Ft. | 12-Ton Truck. | | 15-Ton Truck. | | 20-Ton Truck. | | Span, Ft. | 12-Ton Truck. | | 15-Ton Truck. | | 20-Ton Truck. | |
| | Weight of Wearing Surface, Lb. per Sq. Ft. | | | | | | | Weight of Wearing Surface, Lb. per Sq. Ft. | | | | | |
| | 0 | 100 | 0 | 100 | 0 | 100 | | 0 | 100 | 0 | 100 | 0 | 100 |
| | in. | in. | in. | in. | in. | in. | | in. | in. | in. | in. | in. | in. |
| 2 | 5¾ | 5¾ | 6 | 6 | 6½ | 6½ | 2 | 5¼ | 5¼ | 5½ | 5½ | 5¾ | 5¾ |
| 3 | 6¼ | 6¼ | 6½ | 6½ | 7 | 7 | 3 | 5¾ | 5¾ | 5¾ | 6 | 6½ | 6½ |
| 4 | 6½ | 6½ | 6¾ | 7 | 7¾ | 7¾ | 4 | 6 | 6 | 6¼ | 6¼ | 6¾ | 7 |
| 5 | 6¾ | 7 | 7 | 7½ | 8 | 8¼ | 5 | 6 | 6¼ | 6½ | 6½ | 7 | 7½ |
| 6 | 7 | 7½ | 7½ | 7¾ | 8¼ | 8½ | 6 | 6¼ | 6½ | 6½ | 7 | 7¾ | 7¾ |
| 7 | 7 | 7¾ | 7¾ | 8¼ | 8¾ | 9 | 7 | 6½ | 6¾ | 6¾ | 7½ | 8 | 8¼ |
| 8 | 7½ | 8¼ | 8¼ | 8¾ | 9¼ | 9¾ | 8 | 6¾ | 7¼ | 7½ | 8 | 8½ | 9 |
| 9 | 8 | 8¾ | 8¾ | 9¼ | 10 | 10½ | 9 | 7½ | 8 | 8 | 8½ | 9 | 9½ |
| 10 | 8½ | 9¼ | 9¼ | 10 | 10½ | 11¼ | 10 | 7¾ | 8½ | 8½ | 9 | 9½ | 10 |

Center of reinforcing 1 in. from face of slab for slabs less than 7½ in. thick.
Center of reinforcing 1¼ in. from face of slab for slabs 7½ in. and over, in thickness.
Impact 30 per cent. of live load.
Reinforced as in Table III.

## TABLE III.

### REINFORCEMENT FOR REINFORCED CONCRETE FLOOR SLABS.

The reinforcement given in this table is to be used at the bottom of the slabs figured as simply supported, and at the top and bottom of slabs figured as continuous or partially continuous over the supports. Longitudinal reinforcement ½-in. round or square bars spaced two feet centers.

| Total Thickness, In. | Concrete Outside Center of Steel, In. | Area of Steel per Foot Width, Sq. In. | Weight of Slab, Lb. per Sq. Ft. | Spacing of Bars in Inches. | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Round. | | | | Square. | | | |
| | | | | ⅜ In. | ½ In. | ⅝ In. | ¾ In. | ⅜ In. | ½ In. | ⅝ In. | ¾ In. |
| 5 | 1 | 0.370 | 63 | 3½ | 6¼ | 10 | | 4½ | 8 | 12½ | |
| 5½ | 1 | 0.416 | 69 | 3¼ | 5½ | 9 | | 4 | 7¼ | 11¼ | |
| 6 | 1 | 0.462 | 75 | 2¾ | 5 | 8 | | 3¾ | 6½ | 10 | |
| 6½ | 1 | 0.508 | 81 | 2½ | 4¾ | 7¼ | | 3¼ | 6 | 9¼ | |
| 7 | 1 | 0.554 | 88 | 2¼ | 4¼ | 6¾ | | 3 | 5½ | 8½ | |
| 7½ | 1¼ | 0.578 | 94 | 2¼ | 4 | 6½ | | 3 | 5¼ | 8 | |
| 8 | 1¼ | 0.624 | 100 | 2 | 3¾ | 6 | | 2¾ | 4¾ | 7½ | |
| 8½ | 1¼ | 0.670 | 106 | 2 | 3½ | 5½ | 8 | 2½ | 4½ | 7 | 10 |
| 9 | 1¼ | 0.716 | 113 | | 3¼ | 5¼ | 7½ | | 4¼ | 6½ | 9½ |
| 9½ | 1¼ | 0.762 | 119 | | 3 | 4¾ | 7 | | 4 | 6 | 9 |
| 10 | 1¼ | 0.809 | 125 | | 2¾ | 4½ | 6½ | | 3¾ | 5¾ | 8½ |
| 11 | 1¼ | 0.901 | 138 | | 2½ | 4 | 6 | | 3¼ | 5¼ | 7½ |
| 12 | 1¼ | 0.993 | 150 | | | 3¾ | 5¼ | | 3 | 4¾ | 6¾ |

Interpolate for intermediate slabs.

The Ohio State Highway Commission specifies concrete slabs for different stringer spacings as follows: 5 in. slab for 2 ft. spacing; 6 in. slab for 3 ft. spacing; 6 in. slab for 4 ft. spacing.

Specifications for highway bridges of the state of Nebraska specify slabs made of concrete of a 1-2-4 mix, 6 in. thick reinforced with ½ in. round bars spaced 6 in. centers. The bottom of the concrete to be 1 inch below top of joists.

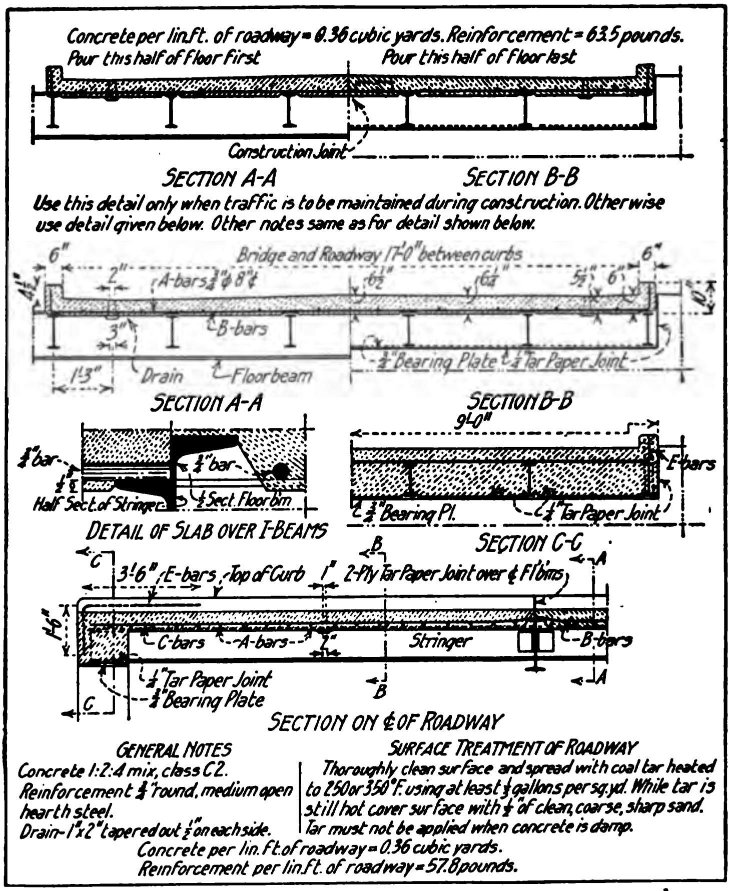



FIG. 1. REINFORCED CONCRETE FLOOR, MICHIGAN HIGHWAY COMMISSION.

The standard reinforced concrete floor used by the Michigan Highway Commission is shown in Fig. 1. The slab is 6½ in. thick at the center and 6 in. thick at the curb. The details of the floor are shown in the cut.

**Miscellaneous Examples.**—The floor in Fig. 3 consists of 3-in. creosoted blocks laid on 5-in. creosoted timbers. The timber blocks were laid in pitch spread on tar paper laid on top of the timbers. Expansion joints were placed at intervals of about 50 ft. transversely, and along the curbs. After a short period of heaving, the pavement reached a constant condition, and has given complete satisfaction.

The timber floor shown in Fig. 4, was used on 23d Street Viaduct, Denver, Colorado, in 1909. The timber wearing surface was replaced in 1917, with a laminated floor made of 2 in. by 4 in.

pine pieces laid edgewise and spiked together. A top wearing surface of asphalt one and one-half inches thick was laid on top of the laminated floor. The results after two years use are satisfactory. Details of timber block floors are shown in Fig. 5 and Fig. 6.

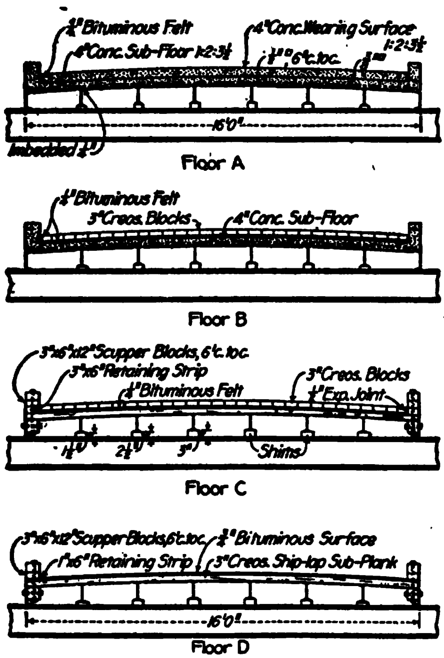



FIG. 2. STANDARD HIGHWAY BRIDGE FLOORS, ILLINOIS HIGHWAY COMMISSION.

**Buckle Plates.**—Buckle plates are made by " dishing " flat plates as in Table 7, Appendix III. The width of the buckle $W$ or length $L$, varies from 2 ft. 6 in. to 5 ft. 6 in. The buckles may be turned with the greater dimension in either dimension of the plate. Several buckles may be put in one plate, all of which must be of the same size and be symmetrically placed. Buckle plates are made $\frac{1}{4}$ in., $\frac{5}{16}$ in., $\frac{3}{8}$ in. and $\frac{7}{16}$ in. thick. Buckle plates should be firmly bolted or riveted around the edges with a maximum spacing of 6 inches, and should be supported transversely between the buckles. The process of buckling distorts the plates and an extra width should be ordered, and the plate should be trimmed after the process is complete. The buckle plates are usually supported on the tops of the stringers, but may be fastened to the bottoms of the stringers. The space above the buckles is filled with concrete which carries the wearing surface. Buckle plates are now seldom used except for special floors and heavy floors where the weight of a reinforced concrete floor would be too great, or where it is necessary to cut down the clearance.

**Plank Floors.**—As long as an excellent grade of timber was available and the concentrated loads were not excessive, timber floors were quite satisfactory when properly constructed. Plank floors should be of white oak, long leaf yellow pine or similar timber, laid transversely. Where two layers of plank are used the lower layer is laid diagonally. Planks should be from 8 in. to 12 in. wide and not less than 3 in. thick. To carry modern auto trucks the plank should have a minimum thickness in inches of three halves the spacing of the stringers in feet. Planks should be laid from $\frac{1}{4}$ in. to $\frac{1}{2}$ in. apart so that water will not be retained, but will run through and will give the planks an opportunity to dry out. Where more than one layer of planks is used a liberal coating of coal tar to the upper side of the lower planks and to the lower side of the upper planks

will materially prolong the life of the floor. The timber in floors made of more than one layer of planks should be creosoted. Each plank should be solidly spiked to each joist with spikes having a length of not less than twice the thickness of the plank, or 6-in. spikes for 3-in. plank and 8-in.

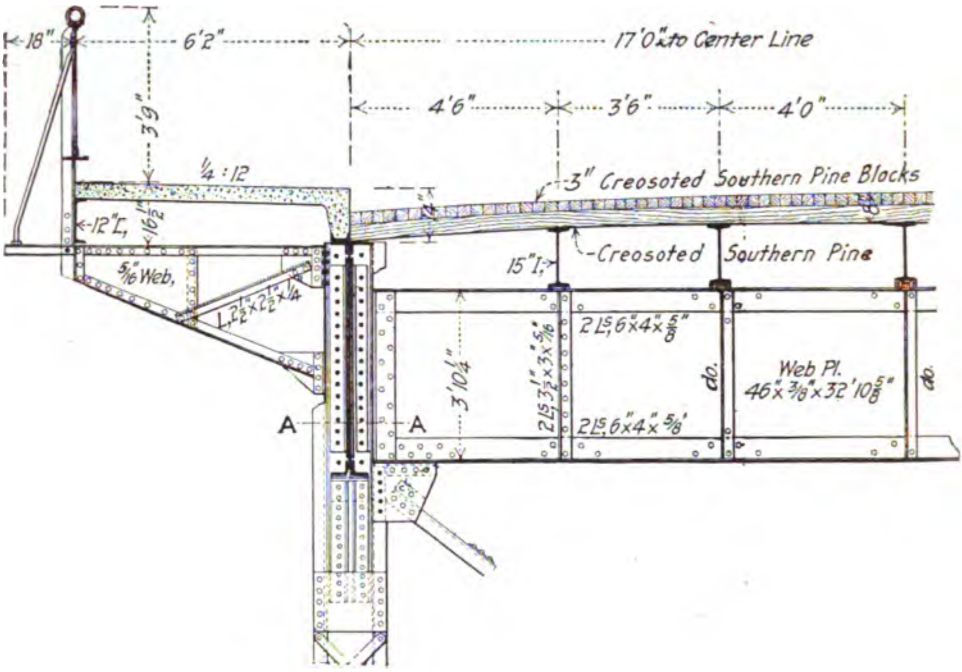



FIG. 3. TIMBER FLOOR, 20TH STREET VIADUCT, DENVER, COLORADO.

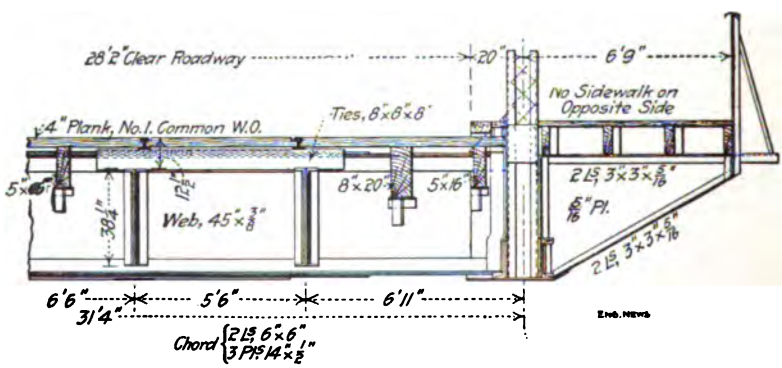



FIG. 4. TIMBER FLOOR, 23RD STREET VIADUCT, DENVER, COLORADO.

spikes for 4-in. plank. Where steel joists are used, spiking strips about 3 in. by 8 in. are bolted to the tops of all joists, or spiking strips 4 in. by 6 in. are bolted to the sides of three lines of joists under each plank length. When the latter method is used the floor planks are fastened to the intermediate joists by bending spikes, driven through the floor plank, around the upper flanges of

the joists. For specifications for plank floors, see the author's "General Specifications for Steel Highway Bridges."

The thickness of plank for different loadings and spans calculated for the allowable stresses required by the author's specifications are given in Table IV.

**Laminated Timber Floor.**—Highway bridge floors are sometimes made by placing 2 in. by 4 in., 2 in. by 6 in., or 3 in. by 8 in. timbers on edge and spiking them together. A waterproof wearing surface is placed on top of the laminated base. The safe spans for a laminated timber floor may be taken the same as for planks 12 inches wide.

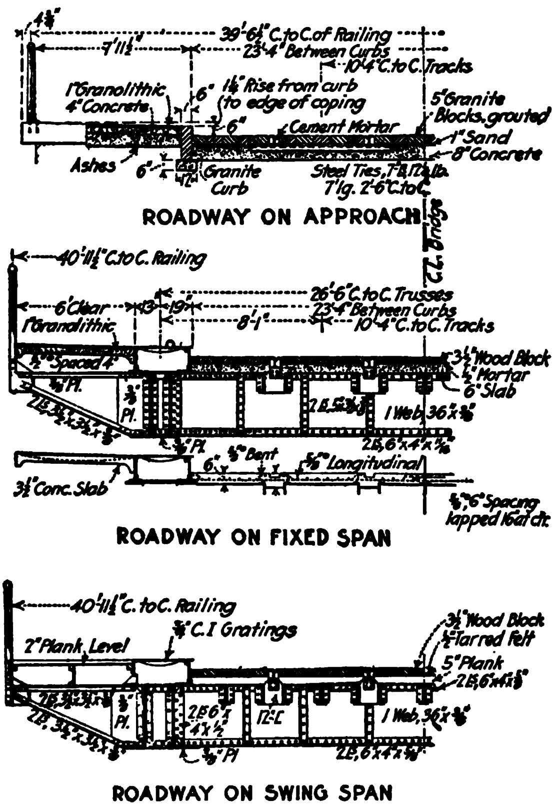



FIG. 5. ROADWAY CONGRESS ST. BRIDGE, CHICAGO, ILL.

The Oregon Highway Commission uses laminated wood floors made of 3 in. by 8 in. timbers placed on edge and spiked together at intervals of not less than 18 in. "The timbers shall preferably be long enough to extend the full width of the roadway, and in no case shall more than two lengths be used in the width of roadway. Every fifth timber shall project ½ in. above the intervening four pieces, to furnish a grip for the water-proof wearing surface."

The Ohio Highway Commission has prepared the following specification for laminated highway bridge floors.

"The lumber shall be oak of the kind and size specified and at least 90 per cent shall have a length equal to the width of the roadway and may be of undressed lumber. If a strip does not extend entirely across the roadway, the splice shall be made over a joist, and splices in adjacent strips shall be staggered. There shall be no variation of more than ¼ inch in the depth of adjacent pieces. Two pieces which are spliced together must be of the same thickness.

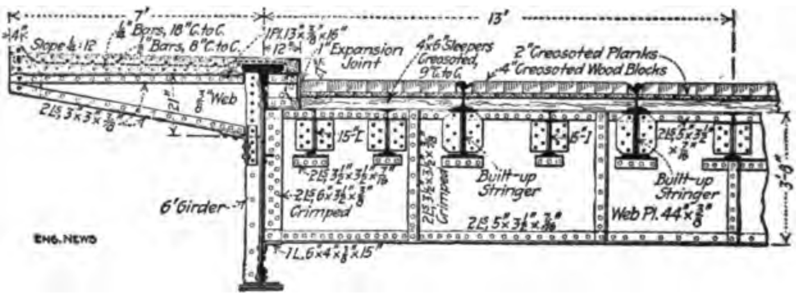



FIG. 6.

"Each strip of the floor shall be placed against the preceding strip laid, the greatest dimension being vertical, and spiked to the preceding strip at each end and between all joists, using 16 d spikes. These spikes shall be omitted on one continuous joint across the roadway about every four feet, so that the floor can be removed in sections in case repairs are needed. Hook bolts along the side of the roadway are to be at the center of each of these sections. Care shall be taken to have each strip vertical and tight against the preceding one, and also to have each strip of the floor bear evenly on all of the joists."

A laminated floor made of 2 in. by 4 in. pine timbers placed on edge and spiked together was used for reflooring 23d Street Bridge, Denver, Colorado. The laminated timber base is covered with an asphalt paving 1½ inches thick.

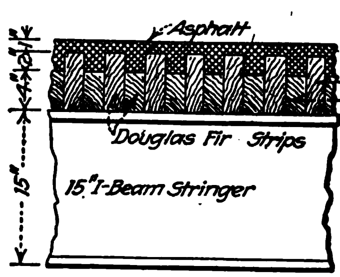



Fig. 7. LAMINATED FLOOR, CHICAGO, ILL.

**Example of Laminated Floor.**—*The laminated floor shown in Fig. 7, has been used on highway bridges in Chicago, Ill. The flooring is built in the field, the strips being spiked to each other with 20 d. spikes spaced about 1 ft. centers, and then face bolted to the stringers with ¾ in. bolts spaced 4 ft. on centers along the bridge and about 5 ft. transversely. The bolts are placed in the grooves. Regulation sheet asphalt was laid 1½ in. over the tops of the strips and granite chips were rolled in with a 5-ton roller. This sheet asphalt covering cost 9 cts. per square foot, delivered hot at the floor and 23 cts. for labor, or a total cost of 32 cts. For future work it is

* Engineering News-Record, November 13–20, 1919.

proposed to use the following mix; bitumen, 8.5 per cent; cement, 9.5 per cent; stone (from $\frac{1}{2}$ in. down to No. 10 mesh) 25 per cent; sand, 57 per cent. On the Erie bascule bridge the strips were supported on steel stringers spaced 2 ft. 8 in. centers. After completion a coal motor truck weighing 14 tons went over the floor at 12 miles per hour without noticeable deflection. The cost per square foot for flooring the Erie St. bridge was 82 cts., divided as follows: 8 cts. for carpenter work at $1.00 per hour, 1 ct. for carpenter foreman, at $8.50 per day; 3 cts. for bolts in place, at 60 cts. per bolt; 38 cts. for select Douglas fir lumber, at $54 per 1,000 ft. b.m.; 32 cts. for asphalt in place, including material and labor. The floor weighs 15 lb. per sq. ft. for lumber and 14 lb. for asphalt, or a total of 29 lb. This floor replaced a floor of 4 in. creosoted blocks laid on 6 in. creosoted planks that weighed 43 lb. per square foot, and cost $1.20 per square foot for material alone. This floor practically eliminates vibration and is practically fireproof and waterproof. While the floors (November, 1919) have only been in service 3 months, the results have proved satisfactory.

## TABLE IV.

### Thickness of 12-inch Floor Plank.

For 8-inch plank add 23 per cent to the thickness of plank.

Thickness in Inches, Actual Size, No Impact.

| Spacing of Joists, In. | 10-Ton Auto Truck. | 12-Ton Auto Truck. | 15-Ton Auto Truck. | 20-Ton Auto Truck. |
|---|---|---|---|---|
| 12 | 2 | 2 | 2 | 2 |
| 15 | $2\frac{3}{8}$ | $2\frac{1}{2}$ | $2\frac{1}{2}$ | $2\frac{5}{8}$ |
| 18 | $2\frac{3}{4}$ | $2\frac{7}{8}$ | 3 | $3\frac{1}{8}$ |
| 21 | 3 | $3\frac{1}{4}$ | $3\frac{3}{8}$ | $3\frac{5}{8}$ |
| 24 | $3\frac{1}{4}$ | $3\frac{1}{2}$ | $3\frac{3}{4}$ | 4 |
| 27 | $3\frac{1}{2}$ | $3\frac{3}{4}$ | 4 | $4\frac{1}{2}$ |
| 30 | $3\frac{3}{4}$ | 4 | $4\frac{3}{8}$ | $4\frac{7}{8}$ |
| 33 | 4 | $4\frac{1}{4}$ | $4\frac{5}{8}$ | $5\frac{1}{4}$ |
| 36 | $4\frac{1}{4}$ | $4\frac{1}{2}$ | $4\frac{7}{8}$ | $5\frac{1}{2}$ |

Allowable Stresses.—Bending stress, 1,500 lb. per sq. in.; bearing across fiber, 400 lb. per sq. in. Minimum thickness of plank allowed by Ketchum's specifications is 3 in.; maximum spacing of joists is 30 in.

**Creosoted Timber Floor.**—Creosoted timber may be used as a sub-floor for a creosoted timber block wearing surface, for a bituminous wearing surface, or may carry a gravel or earth fill, or may have no wearing surface.

**Specifications for Creosoted Timber.**—Timber used for all creosoted floor timbers except blocks shall be first-class oak, long-leaf yellow pine or Oregon fir. It shall be cut from live trees and shall be straight grained, free from shakes, large or loose knots, decayed wood, worm holes or other defects that will impair its strength or durability. It shall be sawed straight and true and shall be full size. All timber shall be impregnated with at least 12 lb. of creosote oil per cubic foot of timber. The creosote oil shall be a pure coal-tar product free from any adulteration. It shall be free from any tar or any petroleum oil or petroleum residue. The specific gravity at 100° F. shall be at least 1.03, but not more than 1.07. The creosote oil shall comply with the specifications of the American Railway Engineering Association for creosote oil. The timber shall be impregnated with creosote oil by the full cell process. The details of the treatment shall comply with the specifications of the American Railway Engineering Association for the treatment of ties with creosote oil.

The timbers for the sub-floor shall be surfaced on one side and one edge, and shall not vary more than $\frac{1}{16}$ in. from the specified thickness. The timbers shall be laid with the surfaced side

down with tight joints, and shall be fastened to the outside spiking strips with two 6-in. lag screws at each end of each plank, and to the intermediate stringers with two spikes in each stringer, the length of the spikes to be at least twice the thickness of the floor planks. The felloe guard shall be bolted to the stringers with $\frac{5}{8}$-in. bolts spaced not more than 5 ft. centers.

**WEARING SURFACES FOR HIGHWAY BRIDGE FLOORS.**—The wearing surface of a highway bridge floor should satisfy the usual conditions for a pavement and in addition should not have an excessive weight; as an increase in dead load on the bridge increases the necessary amount of steel in the floor supports and the trusses and increases the total cost. The most common wearing surfaces will be briefly described.

**Concrete.**—A concrete wearing surface is laid on top of the concrete slab by the Illinois Highway Commission as follows:—The wearing surface shall have a thickness of not less than 4 inches. The lower 2 in. of the wearing surface shall be made of concrete mixed in the proportions of one part Portland cement, 2 parts clean sand and 4 parts clean gravel or broken stone that will pass a $1\frac{1}{2}$-in. ring. The concrete shall be thoroughly mixed in a batch mixer to a jelly-like consistency and shall be placed immediately on the sub-floor slab. Upon this concrete layer shall be immediately laid a 2-in. layer of mortar made by mixing one part Portland cement and 2 parts of clean, coarse sand. The mortar shall be mixed to a jelly-like consistency in a batch mixer and shall be immediately placed upon the freshly laid concrete. Before the mortar has begun to set it shall be finished off with a wood float, and before it has hardened it shall be roughened by brushing with a stiff vegetable brush or broom.

The concrete slab and the concrete wearing surface are commonly laid in one operation, the wearing surface being finished as for a concrete pavement.

**Creosoted Timber Blocks.**—The blocks shall be made of prime sound long-leaf yellow pine or Oregon fir and shall contain no loose knots, worm holes or other defects, and shall be well manufactured. No wood averaging less than 6 rings to the inch, measured radially from the center of the heart shall be used. The blocks shall have a depth as specified, but the depth shall not be less than 3 in. The blocks shall be from 6 to 10 in. long. The width shall be from 3 to 4 in., but the blocks in any contract shall have the same width. A variation of $\frac{1}{16}$ in. in depth and $\frac{1}{8}$ in. in width will be permitted. The width shall be greater or less then the depth by not less than $\frac{1}{4}$ in. The blocks shall be impregnated with creosote oil by the full cell process. The creosote oil and the method of creosoting timber blocks shall be the same as specified for creosoted timber. All creosoted timber blocks shall contain not less than 16 lb. of creosote oil per cubic foot of timber.

*Laying Creosoted Timber Blocks.*—When the creosoted timber blocks are laid on a creosoted timber base, a layer of tar paper shall be laid on the timber base. When creosoted timber blocks are laid on a concrete floor slab, a layer of dry cement mortar made by mixing dry one part of Portland cement and four parts of clean dry sand shall be spread on the dry floor slab. The cement cushion shall be rolled to a thickness of $\frac{1}{2}$ in. As the blocks are laid on the concrete slab the sand and cement shall be moistened by sprinkling and the blocks shall be laid before the cement has had time to set. The blocks shall be laid at right angles to the length of the bridge in parallel lines, with the grain vertical. The blocks shall break joints at least 3 in. Two lines of blocks shall be laid next to the curb with the long dimension of the block parallel to the bridge, and the remainder of the blocks shall be laid at right angles to those blocks. The blocks shall be laid with open joints, $\frac{1}{8}$-in. open joints transversely, $\frac{1}{4}$-in. open joints longitudinally. Expansion joints not less than 1 in. thick the full depth of the block shall be provided along each curb, and transverse joints not less than $\frac{1}{2}$ in. thick shall be provided every 50 ft. in length of the bridge. These joints shall be kept closed until the blocks are all laid, and the space is then to be filled with a bituminous filler. After the blocks have been laid they shall be tamped or rolled to firm bearing. All defective, broken, damaged or displaced blocks shall be removed and replaced with sound blocks. All joints and expansion joints shall then be filled to a depth of two-thirds the depth of the block with a satisfactory bituminous filler. The filler shall not be brittle at 0° F. nor flow at 120° F. The filler shall be applied at a temperature of not less than 300° F. After the first application has

set the joints shall be filled to the proper height with a second coat. Joints shall be filled only in dry weather, when the temperature is not less than 50° F. Before the second coat has hardened a layer of sand ½ in. thick shall be spread on the surface and shall be swept into the joints.

**Bituminous Wearing Surface Floors.**—Bituminous wearing surface floors may be laid on a creosoted timber sub-floor or on a concrete sub-floor.

**Bituminous Wearing Surface on Timber Sub-Floor.**—The bituminous wearing surface may be put on hot by the standard method, or by a cold process. The specifications adopted in 1917 by the Illinois Highway Commission are as follows:

**Bituminous Wearing Surface—Hot Penetration Method. Illinois Highway Commission.**

*Asphalt.*—The asphalt used for bituminous wearing surface shall conform to the following requirements: Asphalt shall have a specific gravity at 25° C. of not less than 0.97 nor more than unity. It shall be soluble in cold carbon disulphide to the extent of at least 98 per cent. Of the total bitumen, not less than 22 per cent nor more than 30 per cent shall be insoluble in 86° B. naphtha. When 20 grams (in a tin dish 2½ in. in diameter and ¾ in. deep with vertical sides) are maintained at a temperature of 163° C. for 5 hours in a N. Y. testing laboratory oven, the evaporation loss shall not exceed 2 per cent and the penetration shall not have been decreased more than 25 per cent. The fixed carbon shall not exceed 16 per cent by weight. The penetration as determined with the Dow machine using a No. 2 needle, 100 gm. weight, 5 seconds time, and a temperature of 25° C. shall be not less than 30 nor more than 50. The asphalt shall contain not to exceed 6 per cent by weight of paraffine scale.

*Aggregate.*—The aggregate shall consist of screened gravel, which shall have been approved by the engineer, dry, free from dust, dirt and clay, and graded in size from ⅜ in. to ¾ in.

*Cleaning Sub-Planking.*—Before placing the wearing surface, the sub-planking shall be thoroughly cleaned from all foreign material and the cracks shall be filled and the plank covered to a depth of approximately ½ in. with asphalt of the character herein specified, which shall be applied at a temperature of not less than 400° F. The sub-planking shall be dry when the asphalt is applied.

*Placing Wearing Surface.*—The gravel shall be spread on the asphalt covering while the same is hot and in a quantity which will just cover the asphalt. The thickness must not exceed that which will be formed by a single layer of the gravel pebbles.

Upon the material thus spread, there shall be poured hot asphalt until the interstices are all filled, the asphalt being at a temperature of not less than 400° F.

Upon the layer of asphalt thus poured there shall be spread a second layer of gravel which shall not exceed the thickness of a single layer of pebbles, but which must be spread in sufficient quantity to cover completely the layer of asphalt.

Upon the layer of gravel thus spread there shall be poured hot asphalt until all the interstices are filled, the asphalt having a temperature of not less than 400° F.

*Finish.*—The surface shall then be covered with a layer of pebbles just sufficient to cover the asphalt, the pebbles to be well rolled or tamped into the asphalt and the surface finally covered with coarse sand sufficient to take up any free asphalt. After the surface has stood for one day, it may be opened to traffic.

**Bituminous Wearing Surface—Cold Mixing Method, using an Asphalt Emulsion. Illinois Highway Commission.**

*Asphalt Emulsion.*—The emulsion shall consist of asphalt, water and fatty or resin soap thoroughly emulsified. It shall conform to the following requirements:

| | |
|---|---|
| Total bitumen | Not less than 60.0 per cent |
| Specific gravity of dehydrated material | Not less than 1.000 |
| Penetration of dehydrated material, 25° C., 100 gm., 5 sec. | 150 to 200 |

*Total Bitumen.*—The total bitumen shall be considered as being 100 minus the sum of the percentages of water, of fatty or resin acids, of organic matter insoluble in carbon disulphide other than fatty or resin acids from the soap, or mineral matter (ash), and of ammonia.

For percentages of water, fatty or resin acids, organic matter insoluble in carbon disulphide, mineral matter (ash), and ammonia, see United States Department of Agriculture Bulletin 314, p. 41.

*Specific Gravity.*—Standardized pycnometers, United States Department of Agriculture Bulletin 314, p. 4.

*Penetration.*—A. S. T. M. Stand. Test D 5–16.

*Aggregate.*—The aggregate shall consist of crushed stone chips uniformly graded from ¾ in. down to dust with all dust removed, to which shall be added sufficient sand to fill all remaining voids, but not to exceed 20 per cent of the volume of the aggregate.

*Cleaning Sub-Planking.*—Before placing the wearing surface, the sub-planking shall be thoroughly cleaned from all foreign material and all cracks shall be filled with wood strips or oakum.

*Mixing Materials.*—The aggregate and the asphalt emulsion shall be mixed cold in the proportions of 1 gal. of emulsion to 1 cu. ft. of aggregate. To facilitate mixing, water to the extent of 20 per cent may be added to the emulsion. The proportions given above for mixing the aggregate and the emulsion are based on the undiluted emulsion. The mixing shall be done on a tight mixing board or in a batch concrete mixer, and shall continue until all particles of the aggregate are thoroughly coated.

*Placing Wearing Surface.*—After mixing, the material shall be spread upon the roadway in sufficient quantity to provide a thickness of ¾ in., after rolling or tamping.

*Finish.*—After the material has been rolled or tamped smooth and to a uniform thickness of ¾ in., the surface shall be given a paint coat of the emulsion applied at the rate of ½ gal. per sq. yd., and then shall be covered with coarse sand sufficient to take up any free asphalt and to fill all voids in the surface. After the surface has stood for one day, it may be opened to traffic.

**Bituminous Pavement on Concrete.**—A bituminous wearing surface may be laid as on the creosoted plank sub-floor, or the wearing surface may be laid according to the following standard method. The concrete shall be dry and thoroughly clean. A bituminous wearing surface two inches thick is applied as follows: The aggregate consists of broken stone or gravel passing a one-inch screen with the dust screened out to which is added sand equal to about one-quarter to one-half the volume of the stone. The aggregates shall be heated and mixed with the bituminous material in a mechanical mixer or by hand with hot shovels. The asphalt shall be mixed not less than 20 gallons to the cubic yard of aggregate at a temperature of 350° to 400° F. The mixture shall be applied hot to the concrete surface and shall be raked with hot hoes or rakes and rolled with a roller weighing not less than 5 tons. After the surface has been rolled a layer of hot asphalt shall be applied and a layer of coarse sand rolled into the hot asphalt.

**Examples of Highway Bridge Floors.**—The following examples of highway bridge floors specified by different highway commissions are of interest.

The Illinois Highway Commission uses the following standard floors: (1) A reinforced concrete sub-floor 4 in. thick, and a concrete wearing surface 4 in. thick, weight 100 lb. per sq. ft.; (2) a reinforced concrete sub-floor 4 in. thick and a creosoted timber block wearing surface 3 in. thick, weight 65 lb. per sq. ft.; (3) a creosoted plank sub-floor 3 in. thick and a wearing surface of creosoted timber blocks 3 in. thick, weight 32 lb. per sq. ft.; and (4) a creosoted timber ship lap floor 3 in. thick and a wearing surface of creosoted timber blocks 3 in. thick, weight 26 lb. per sq. ft. For details of Illinois Highway Commission's standard floors see Fig. 2.

The Michigan Highway Commission uses the following surface treatment on concrete floor slabs. The surface of the concrete is thoroughly cleaned and ⅓ of a gallon of coal tar per sq. yd., heated to a temperature of 250° to 350° F. is spread over the slab. While the tar is hot the surface is evenly covered with a layer ½ in. thick of clean, sharp, coarse sand.

The Wisconsin Highway Commission does not specify a wearing coat on top of concrete floor slabs.

The Iowa Highway Commission uses either a 3 in. fill of gravel or a creosoted block floor 3 in. thick. Concrete slabs are covered with a bituminous coating made by applying to the clean dry slab ⅓ of a gallon of hot tar per sq. yd. A layer of coarse dry sand is heated and sifted on top of the tar.

For additional examples of highway bridge floors and data on concentrated loads on bridges and details of the design of highway bridge floors, see paper entitled "Highway Bridge Floors," by Professor Charles M. Spofford, Proceedings Society Western Pennsylvania, Vol. 31, pp. 727–826.

**Cost of Floors.**—The costs of highway bridge floors were estimated by Mr. Clifford Older, bridge engineer, Illinois Highway Commission in 1915 as follows: Concrete in sub-floors including reinforcing steel, $12.00 per cu. yd.; concrete wearing surface, 4 in. thick, $0.90 per sq. yd.; creosoted sub-plank (12-lb. treatment) in place, $70 per thousand feet B. M.; creosoted blocks 3 in. thick, in place, $1.80 per sq. yd.; bituminous gravel wearing surface, ¾ in. thick, $0.60 per sq. yd. The weights and costs of the Illinois Highway Commission standard floors were as follows: concrete sub-floor 4 in. thick and concrete wearing surface 4 in. thick, weighs 100 lb. per sq. ft.,

and cost $2.95 per sq. yd.; concrete sub-floor 4 in. thick, and creosoted blocks 3 in. thick, weighs 65 lb. per sq. ft., and cost $3.25 per sq. yd.; creosoted plank sub-floor 3 in. thick, and creosoted blocks 3 in. thick, weighs 32 lb. per sq. ft., and cost $4.10 per sq. yd.; creosoted plank sub-floor 3 in. thick, and bituminous wearing surface ¾ in. thick, weighs 26 lb. per sq. ft., and cost $3.00 per sq. yd.

**DESIGN OF STRINGERS.**—Stringers or joists support the floor and in turn are supported by the floorbeams. The joists may be supported on the tops of the floorbeams or may be framed into the floorbeam by the use of connection angles. Where concrete floors are used the steel joists should either be supported on the tops of the floorbeams or if framed into the floorbeams should have the upper flanges of the beams coped so that the tops of the joists will be on the same level as the floorbeams. The loads carried by the joists are (1) the dead load which is made up of the weight of the joists, the floor slab and the wearing surface; (2) a uniform live load, or a concentrated moving load. The uniform live load and the concentrated moving loads are the same as the loads used in designing the floor slabs, but the distribution of the concentrated load is not the same. The distribution of the moving concentrated load to the joists as specified by different highway commissions and others, and by the author have already been given.

**Steel Stringers.**—The sizes of steel I-beams of minimum weights required for stringers with different spacings to carry a dead load of 100 lb. per sq. ft. and a 20-ton auto truck with 30 per cent impact or a live load of 125 lb. per sq. ft. with 30 per cent impact are given in Fig. 8; and to carry a dead load of 100 lb. per sq. ft. and a 15-ton auto truck with 30 per cent impact or a live load of 100 lb. per sq. ft. with 30 per cent impact are given in Fig. 9. The sizes of steel I-beams of minimum weights required to carry a dead load of 100 lb. per sq. ft. and a 15-ton auto truck without impact or a live load of 100 lb. per sq. ft. without impact are given in Fig. 10. The steel stringers used by the Wisconsin Highway Commission to carry a 15-ton road roller without impact, and the steel stringers used by the Iowa Highway Commission to carry a 15-ton traction engine without impact are practically the same as those given in Fig. 10.

TABLE V.

SPACING OF TIMBER STRINGERS OR JOISTS.

Calculated for 20-ton Auto Truck, Without Impact.

| Nominal Size of Joists, In. | Maximum Spacing in Feet for Different Spans in Feet. | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 |
| 3 × 10...... | 0.7 | 0.7 | 0.6 | | | | | |
| 4 × 10...... | 0.9 | 0.9 | 0.8 | | | | | |
| 3 × 12...... | 0.8 | 0.8 | 0.8 | 0.7 | | | | |
| 4 × 12...... | 1.1 | 1.1 | 1.1 | 1.0 | | | | |
| 3 × 14...... | 1.0 | 1.0 | 1.0 | 1.0 | 0.8 | | | |
| 4 × 14...... | 1.3 | 1.3 | 1.3 | 1.3 | 1.1 | 1.0 | | |
| 6 × 14...... | 2.0 | 2.0 | 2.0 | 2.0 | 1.7 | 1.5 | 1.3 | 1.2 |
| 4 × 16...... | 1.5 | 1.5 | 1.5 | 1.5 | 1.5 | 1.3 | 1.2 | 1.0 |
| 6 × 16...... | 2.2 | 2.2 | 2.2 | 2.2 | 2.2 | 2.0 | 1.7 | 1.5 |

The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists in feet divided by four feet.

Joists were designed for allowable stresses as follows: Cross-bending, 1,500 lb. per sq. in.; bearing across the grain 400 lb. per sq. in.; longitudinal shear 140 lb. per sq. in.

Spacing of joists for spans to left of heavy line are determined by longitudinal shear.

**Timber Joists.**—The sizes of timber stringers or joists for different spacings and spans to carry a 20-ton auto truck are given in Table V; to carry a 15-ton auto truck in Table VI, and to carry a 10-ton auto truck in Table VII. The timber joists were designed for the following unit stresses, to be used without impact: Allowable bending stress, 1,500 lb. per sq. in.; allowable bearing across the grain, 400 lb. per sq. in.; allowable longitudinal shear in beams, 140 lb. per sq. in. The maximum spacings of timber joists for short spans are determined by the longitudinal shear.

## TABLE VI.

### SPACING OF TIMBER STRINGERS OR JOISTS.

Calculated for 15-ton Auto Truck, Without Impact.

| Nominal Size of Joists, In. | Maximum Spacing in Feet for Different Spans in Feet. | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 |
| 3 × 10...... | 1.0 | 1.0 | 0.8 | | | | | |
| 4 × 10...... | 1.3 | 1.3 | 1.1 | 0.9 | | | | |
| 3 × 12...... | 1.1 | 1.1 | 1.1 | 1.0 | | | | |
| 4 × 12...... | 1.6 | 1.6 | 1.6 | 1.4 | 1.2 | 1.0 | | |
| 3 × 14...... | 1.4 | 1.4 | 1.4 | 1.4 | 1.2 | 1.0 | | |
| 4 × 14...... | 1.9 | 1.9 | 1.9 | 1.9 | 1.6 | 1.4 | 1.2 | 1.1 |
| 6 × 14...... | 2.8 | 2.8 | 2.8 | 2.8 | 2.4 | 2.0 | 1.8 | 1.6 |
| 4 × 16...... | 2.1 | 2.1 | 2.1 | 2.1 | 2.1 | 1.8 | 1.6 | 1.5 |
| 6 × 16...... | 3.1 | 3.1 | 3.1 | 3.1 | 3.1 | 2.7 | 2.4 | 2.2 |

The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists in feet divided by four feet.

Joists were designed for allowable stresses as follows: Cross-bending, 1,500 lb. per sq. in.; bearing across the grain, 400 lb. per sq. in.; longitudinal shear, 140 lb. per sq. in.

Spacing of joists for spans to left of heavy line are determined by longitudinal shear.

## TABLE VII.

### SPACING OF TIMBER STRINGERS OR JOISTS.

Calculated for 10-ton Auto Truck, Without Impact.

| Nominal Size of Joists In. | Maximum Spacing in Feet for Different Spans in Feet. | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 6 | 8 | 10 | 12 | 14 | 16 | 18 | 20 |
| 3 × 10...... | 1.4 | 1.4 | 1.2 | 1.0 | 0.9 | | | |
| 4 × 10...... | 2.0 | 2.0 | 1.7 | 1.4 | 1.2 | 1.0 | | |
| 3 × 12...... | 1.8 | 1.8 | 1.8 | 1.5 | 1.3 | 1.1 | 1.0 | |
| 4 × 12...... | 2.4 | 2.4 | 2.4 | 2.0 | 1.8 | 1.5 | 1.4 | 1.2 |
| 3 × 14...... | 2.0 | 2.0 | 2.0 | 2.0 | 1.8 | 1.5 | 1.4 | 1.2 |
| 4 × 14...... | 2.8 | 2.8 | 2.8 | 2.8 | 2.4 | 2.1 | 1.9 | 1.7 |
| 6 × 14...... | 4.1 | 4.1 | 4.1 | 4.1 | 3.5 | 3.1 | 2.8 | 2.5 |
| 4 × 16...... | 3.2 | 3.2 | 3.2 | 3.2 | 3.2 | 2.8 | 2.5 | 2.2 |
| 6 × 16...... | 4.7 | 4.7 | 4.7 | 4.7 | 4.7 | 4.1 | 3.6 | 3.3 |

The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists in feet divided by four feet.

Joists were designed for allowable stresses as follows: Cross-bending, 1,500 lb. per sq. in.; bearing across the grain, 400 lb. per sq. in.; longitudinal shear, 140 lb. per sq. in.

Spacing of joists for spans to left of heavy line are determined by longitudinal shear.

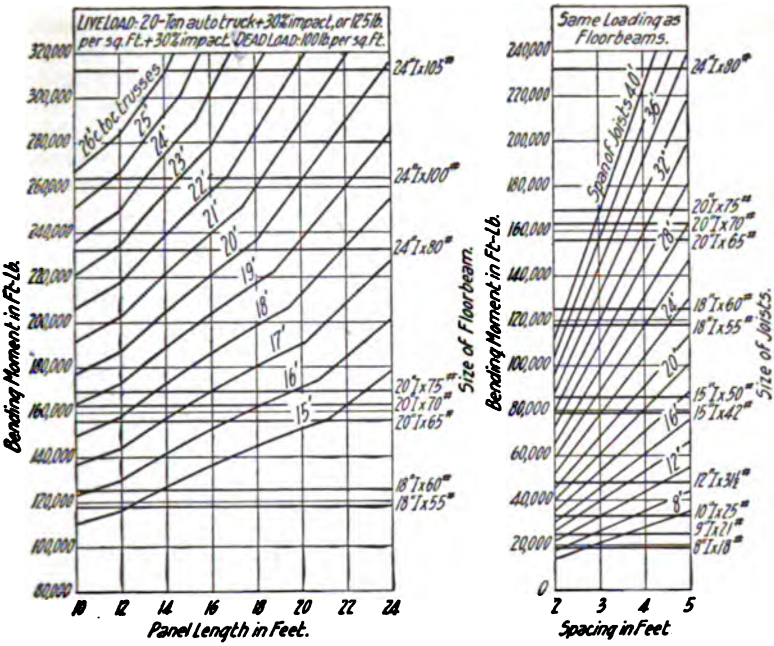



FIG. 8. BENDING MOMENTS IN FLOORBEAMS AND STRINGERS FOR 20-TON AUTO TRUCK. (30 PER CENT IMPACT). CONCRETE FLOOR.

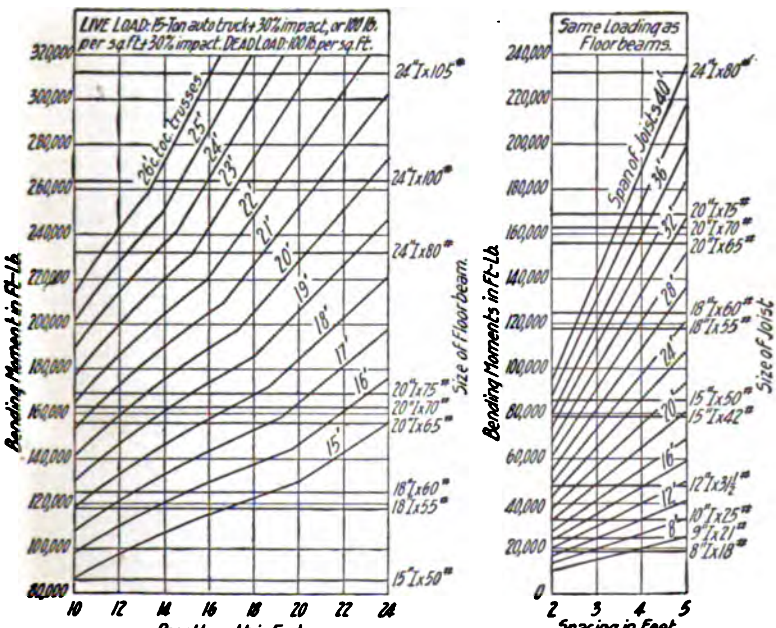



FIG. 9. BENDING MOMENTS IN FLOORBEAMS AND STRINGERS FOR 15-TON AUTO TRUCK. (30 PER CENT IMPACT). CONCRETE FLOOR.

**DESIGN OF FLOORBEAMS.**—The floor loads may be carried to the floorbeams by means of stringers or joists, or the loads may be carried to the floorbeams directly by the floor slabs. The loads carried by the floorbeams consist of (1) the dead load which is the weight of the floor system; (2) a uniform live load or a concentrated moving load. The uniform live loads are the same as the uniform live loads used in designing the floor slabs and stringers, but the distribution of the concentrated moving load is not the same as for either the floor slabs or the stringers. The distribution of the moving concentrated load to floorbeams as specified by different highway commissions and others, and by the author have been given in Chapter IX.

**Steel I-Beam Floorbeams.**—The sizes of steel I-beams required for floorbeams for panel lengths of 10 ft. to 24 ft. and widths center to center of trusses or girders of 15 ft. to 26 ft., to carry a dead load of 100 lb. per sq. ft., and a 20-ton auto truck with 30 per cent impact, or a uniform live load of 125 lb. per sq. ft. with 30 per cent impact are given in Fig. 8; while the floorbeams required to carry a 15-ton auto truck with 30 per cent impact, or a uniform live load of 100 lb. per sq. ft. with 30 per cent impact are given in Fig. 9. It will be noted that the uniform live load controls for wide roadways or for long panels.

For a bridge 17 ft. center of trusses and 18 ft. panels, from Fig. 8 the required floorbeam is a 24 in. I @ 80 lb., while from Fig. 9 the required floorbeam is a 20 in. I @ 70 lb.

The sizes of steel I-beams required for floorbeams for panel lengths of 10 ft. to 24 ft., and widths center to center of trusses or girders of 15 ft. to 26 ft., to carry a dead load of 100 lb. per sq. ft. and a 15-ton auto truck without impact, or a uniform live load of 100 lb. per sq. ft. without impact are given in Fig. 10. These are practically the floorbeams required by the specifications of the Illinois, Iowa, and Wisconsin Highway Commissions. Steel stringers for the same loading are given in Fig. 10.

The bending moments for the design of built-up floorbeams may be obtained from Fig. 8, Fig. 9, or Fig. 10.

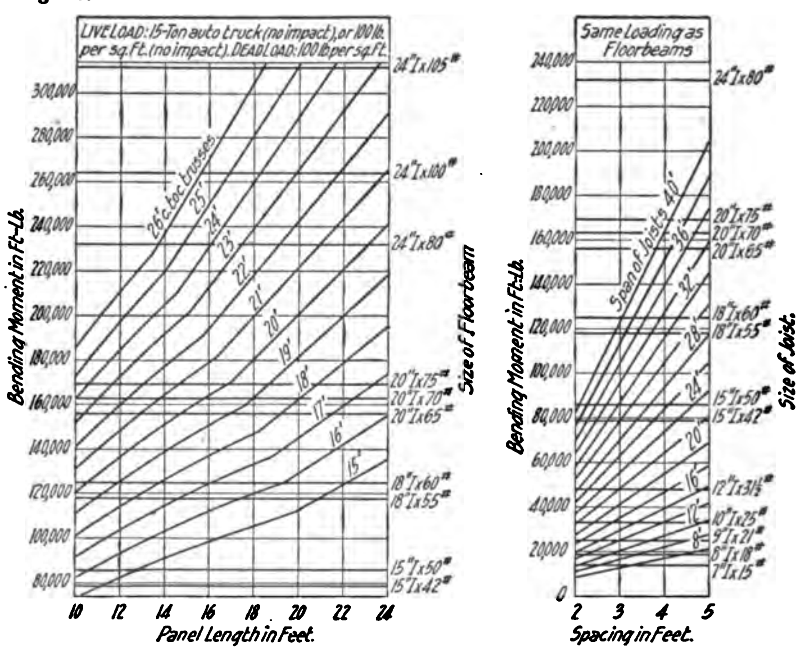



FIG. 10. BENDING MOMENTS IN FLOORBEAMS AND STRINGERS FOR 15-TON AUTO TRUCK. (NO IMPACT). CONCRETE FLOOR.

# CHAPTER XI.

## Design of Beam Highway Bridges.

**Introduction.**—Beam bridges are made by placing steel beams side by side with the ends resting on the abutments. The roadway floor may have a concrete sub-floor with an earth fill wearing surface, a timber block wearing surface, an asphalt or bituminous wearing surface, or a concrete wearing surface; or a plank floor may be used. The spacing of the beams depends upon the load to be carried and upon the type of floor. An old rule for the thickness of oak floor planks was that the floor should have at least one inch in thickness for each foot of spacing of joists or stringers. With the modern auto truck a better rule is that the floor planks shall have three inches in depth for each two feet of spacing of joists or stringers, with a minimum thickness of 3 inches. Joists or stringers with plank floors should not have a greater spacing than 3 feet. The thickness of concrete slabs required to carry a given load is practically a constant from 2 ft. to 4 ft. or 5 ft., and it is therefore commonly economical to use a larger spacing for joists or stringers when used to support a concrete sub-floor than when used to support a plank floor. The outside beams should be the same size as the intermediate beams. It is commonly specified that rolled beams when used for stringers shall have a depth not less than one-thirtieth of the span.

Standard steel beam bridges, as designed by the American Bridge Company, are shown in Fig. 1 and Fig. 2. The details of both bridges are the same with the exception of the fence. Angle cross-braces are used on both bridges. The beams rest directly on the bridge seat of the abutment and not on wall channels as is a more common practice, although a channel is sometimes laid on the bridge seat with the legs turned down to carry the beams. The gas pipe rail in Fig. 1 is much cheaper than the lattice rail in Fig. 2.

In the place of the spiking strips on the tops of the beams, as shown in Figs. 1 and 2, spiking strips are sometimes bolted on the sides of the channels and the center I beam, or two channels are used for the center beam with the spiking strip bolted between them. The floor planks are spiked to these spiking strips, and are fastened to the other beams by clinching spikes, which have been driven through the planks, around the top flanges of the beams.

The maximum span for beam bridges is usually given as 40 feet. A better limit for beam spans is 32 feet. Riveted truss bridges should be used for spans of 32 feet and upwards for country bridges, and plate girders for heavy city bridges. Riveted bridges for spans of, say, 40 feet are more economical than beam bridges and will give fully as great a length of service. The ends of beam bridges should always be supported on masonry abutments.

Details of a standard beam bridge with a concrete floor as adopted by the Wisconsin Highway Commission are shown in Fig. 3. The beams are fastened to the wall plate channels by means of U-bolts. Drains through the floor slab at the gutter are provided at distances of 10 ft. or 12 ft. The reinforced concrete slab is 6 in. thick at the center and 5 in. thick at the curbs, reinforced transversely with $\frac{1}{2}$ in. square twisted bars spaced 6 in. centers, with center of bars 1 in. from bottom of slab, and the longitudinal bars are $\frac{1}{2}$ in. square twisted, spaced two between adjacent joists. The posts of the rail are anchored to the concrete slab floor. Channels are used for the outside stringers in the place of I-beams, which is the better practice. The sizes of I-beams used by the Wisconsin Highway Commission for different spans are given in Table I. These beam spans are designed for a 15-ton road roller, not considering impact.

Details of a beam bridge with concrete floor and fence as designed by the Iowa Highway Commission are given in Fig. 4. The sizes of I-beams, quantities of material and other data for

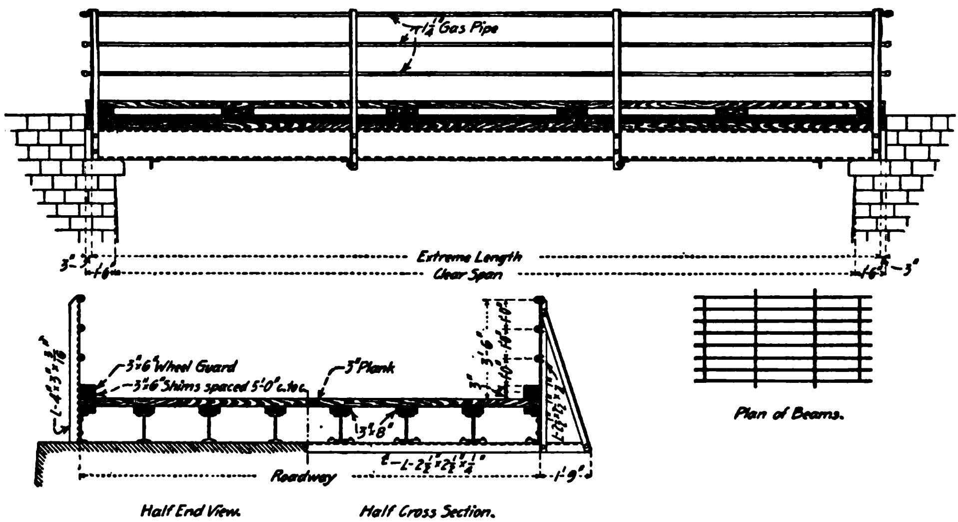



FIG. 1. BEAM BRIDGE. AMERICAN BRIDGE COMPANY.

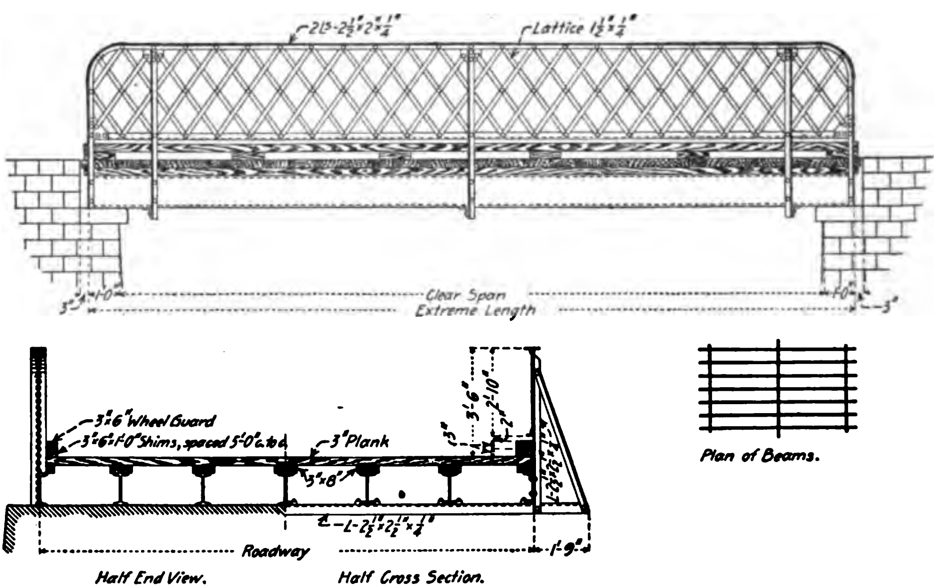



FIG. 2. BEAM BRIDGE. AMERICAN BRIDGE COMPANY.

beam spans from 16 ft. to 32 ft. are given in the cut. It will be noted that I-beams are used for the outside stringers in this design. The outside stringers are wrapped with wire mesh and are encased in concrete. Weep holes 2 in. in diameter spaced 4 ft. apart are provided on each side of the bridge.

Details of a beam bridge with concrete floor and an angle rail, as designed by the Iowa Highway Commission are given in Fig. 5. The sizes of I-beams and channels for spans of 16 ft. to 32

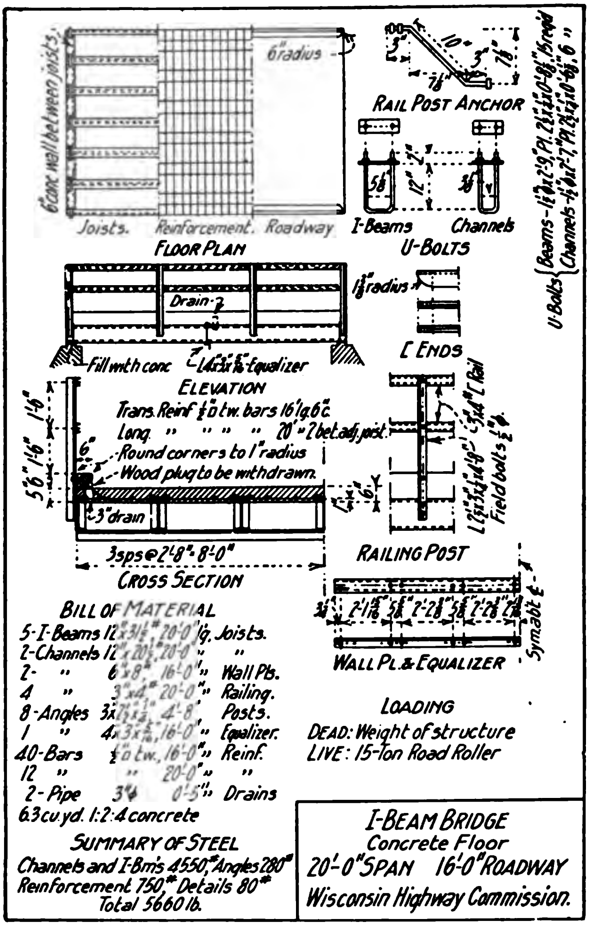



FIG. 3. STANDARD BEAM BRIDGE. WISCONSIN HIGHWAY COMMISSION.

ft. are given in the cut. This bridge is also constructed with a gas pipe fence in place of the angle fence. Estimated quantities of steel and concrete in beam spans of this type are given in Table II.

The depths of I-beams for different spans for beam spans with concrete floors designed to carry a 20-ton, a 15-ton, or a 10-ton auto truck with 30 per cent impact are given in Table III. The minimum weights of I-beams are to be used in each case. These beam spans were designed for a dead load of 100 lb. per sq. ft. in addition to the concentrated live loads. The outside beams are to be the same as the intermediate beams. The thickness of the slabs for the different loadings

## TABLE I.

STEEL I-BEAM BRIDGES. WISCONSIN HIGHWAY COMMISSION.
Channels on outside. Weight includes railing.

| Span, Ft. | 16 Feet Roadway. | | | 18 Ft. Roadway. | | | 20 Ft. Roadway. | | |
|---|---|---|---|---|---|---|---|---|---|
| | No. Beams and Channels. | Size I-Beams, In. Lb. | Weight Structural Steel, Lb. | No. Beams and Channels. | Size I-Beams, In. Lb. | Weight Structural Steel, Lb. | No. Beams and Channels. | Size I-Beams, In. Lb. | Weight Structural Steel, Lb. |
| 10 | 8 | 8—18 | 1,900 | 9 | 8—18 | 2,120 | 10 | 8—18 | 2,335 |
| 12 | 8 | 8—18 | 2,200 | 9 | 8—18 | 2,450 | 10 | 8—18 | 2,700 |
| 14 | 8 | 9—21 | 2,800 | 9 | 9—21 | 3,130 | 10 | 9—21 | 3,465 |
| 16 | 8 | 9—21 | 3,185 | 9 | 9—21 | 3,560 | 10 | 9—21 | 3,930 |
| 18 | 8 | 10—25 | 4,030 | 9 | 10—25 | 4,505 | 10 | 10—25 | 5,000 |
| 20 | 7 | 12—31½ | 4,810 | 8 | 12—31½ | 5,600 | 9 | 12—31½ | 6,285 |
| 22 | 8 | 12—31½ | 6,050 | 9 | 12—31½ | 6,790 | 10 | 12—31½ | 7,545 |
| 24 | 8 | 12—31½ | 6,435 | 9 | 12—31½ | 7,350 | 10 | 12—31½ | 8,160 |
| 26 | 7 | 15—42 | 8,275 | 8 | 15—42 | 9,420 | 9 | 15—42 | 10,570 |
| 28 | 8 | 15—42 | 10,045 | 9 | 15—42 | 11,275 | 10 | 15—42 | 12,510 |
| 30 | 8 | 15—42 | 10,715 | 9 | 15—42 | 12,025 | 10 | 15—42 | 13,350 |
| 32 | 7 | 18—55 | 12,050 | 8 | 18—55 | 13,930 | 9 | 18—55 | 15,750 |
| 34 | 7 | 18—55 | 12,825 | 8 | 18—55 | 15,760 | 9 | 18—55 | 16,685 |
| 36 | 8 | 18—55 | 15,530 | 9 | 18—55 | 17,570 | 10 | 18—55 | 19,615 |
| 38 | 8 | 18—55 | 16,350 | 9 | 18—55 | 18,405 | 10 | 18—55 | 20,655 |

| | 16-ft. Rdwy. | 18-ft. Rdwy. | 20-ft. Rdwy. |
|---|---|---|---|
| Weight in lb. of reinforcing per lineal foot.... | 40 | 44 | 48 |
| Cu. yd. concrete per lineal foot.............. | 0.32 | 0.36 | 0.40 |

## TABLE II.

ESTIMATED QUANTITIES FOR STANDARD BEAM SPANS. IOWA HIGHWAY COMMISSION.

| Span, Ft. | Structural Steel. | | | Reinforced Concrete Floor. | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | Roadway. | | | 16 Ft. Roadway. | | 18 Ft. Roadway. | | 20 Ft. Roadway. | |
| | 16 Ft. | 18 Ft. | 20 Ft. | Concrete. | Steel. | Concrete. | Steel. | Concrete. | Steel. |
| | lb. | lb. | lb. | cu. yd. | lb. | cu. yd. | lb. | cu. yd. | lb. |
| 16 | 3,370 | 3,780 | 3,800 | 5.6 | 600 | 6.3 | 680 | 7.0 | 740 |
| 18 | 4,280 | 4,810 | 4,820 | 6.2 | 670 | 7.0 | 750 | 7.7 | 820 |
| 20 | 4,720 | 5,300 | 5,320 | 6.8 | 730 | 7.6 | 830 | 8.5 | 900 |
| 22 | 6,340 | 7,130 | 7,150 | 7.4 | 800 | 8.3 | 900 | 9.2 | 990 |
| 24 | 6,840 | 7,690 | 7,710 | 8.0 | 870 | 9.0 | 980 | 10.0 | 1,070 |
| 26 | 7,330 | 8,240 | 8,260 | 8.6 | 930 | 9.7 | 1,050 | 10.7 | 1,150 |
| 28 | 10,570 | 11,870 | 11,880 | 9.2 | 1,000 | 10.4 | 1,120 | 11.5 | 1,230 |
| 30 | 11,240 | 12,620 | 12,640 | 9.8 | 1,060 | 11.0 | 1,200 | 12.2 | 1,310 |
| 32 | 11,910 | 13,370 | 13,390 | 10.4 | 1,130 | 11.7 | 1,270 | 13.0 | 1,390 |

Standard angle railing for wing walls as shown in Fig. 12.
Rails ∠s 2½" × 2½" × ¼" × 5'-9". Top of rail 3'-2" above grade. Post ∠s 3" × 3" × ¼" × 4'-3".
Weight of rails and posts for one wing = 90 lb.

are given in Table I and Table II, Chapter X. The depths of I-beams for different spans for beam spans with plank floors designed to carry a 20-ton, a 15-ton, or a 10-ton auto truck with 30 per cent impact are given in Table IV. The minimum weights of I-beams are to be used in each

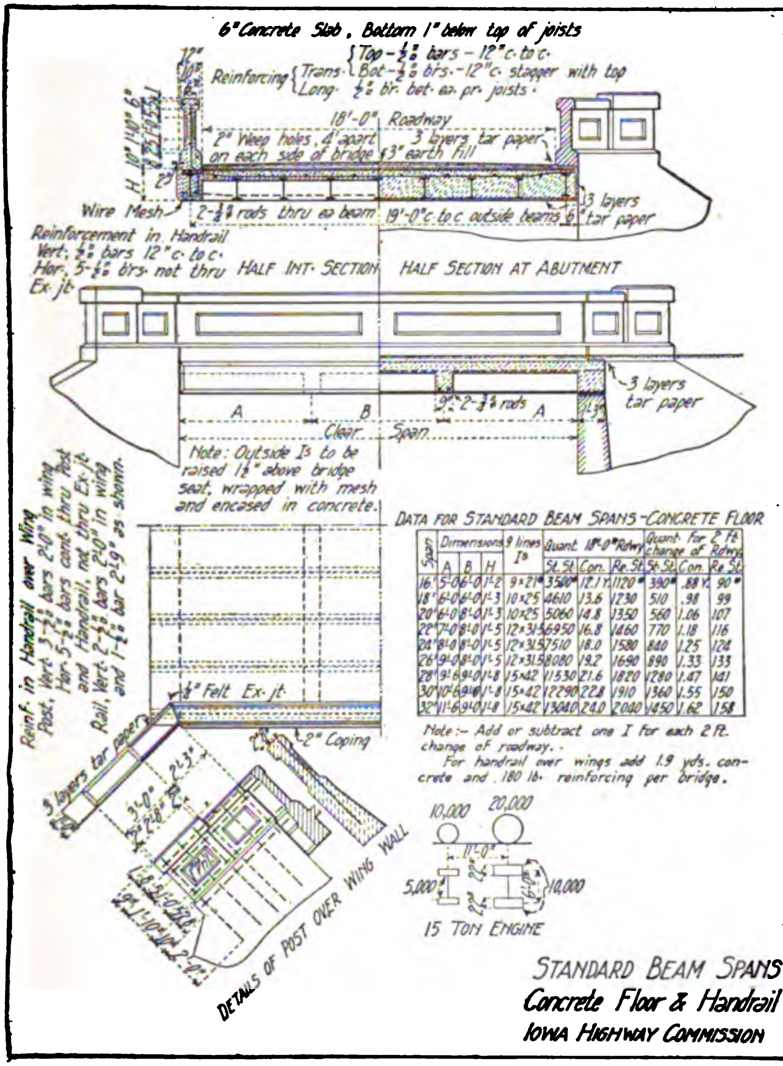



FIG. 4. STANDARD BEAM BRIDGE. IOWA HIGHWAY COMMISSION.

case. In the calculations it was assumed that one stringer carried that fraction of the concentrated load equal to the fraction found by dividing the stringer spacing by 6 feet for concrete floors, and by dividing the stringer spacing by 4 feet for plank floors. It will be noted that heavier beams are required for short spans with plank floors than with concrete floors.

## TABLE III.

DEPTH IN INCHES OF I-BEAMS FOR DIFFERENT SPACINGS AND SPANS REQUIRED TO CARRY 20-TON, 15-TON AND 10-TON AUTO TRUCKS AND 30 PER CENT IMPACT. DEAD LOAD 100 LB. PER SQ. FT. MINIMUM WEIGHTS OF I-BEAMS ARE USED.

| Concrete Floor. | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Span, Ft. | 20-Ton Auto Truck. | | | 15-Ton Auto Truck. | | | 10-Ton Auto Truck. | | |
| | Spacing, Ft. | | | Spacing, Ft. | | | Spacing, Ft. | | |
| | 2 | 3 | 4 | 2 | 3 | 4 | 2 | 3 | 4 |
| 10 | 8 | 10 | 12 | 7 | 9 | 10 | 6 | 8 | 9 |
| 12 | 9 | 10 | 12 | 8 | 9 | 10 | 7 | 8 | 9 |
| 14 | 10 | 12 | 15 | 9 | 10 | 12 | 8 | 9 | 10 |
| 16 | 10 | 12 | 15 | 9 | 12 | 12 | 8 | 10 | 12 |
| 18 | 12 | 15 | 15 | 10 | 12 | 15 | 9 | 10 | 12 |
| 20 | 12 | 15 | 18 | 10 | 15 | 15 | 9 | 12 | 12 |
| 22 | 12 | 15 | 18 | 12 | 15 | 15 | 10 | 12 | 15 |
| 24 | 15 | 15 | 18 | 12 | 15 | 18 | 10 | 12 | 15 |
| 26 | 15 | 18 | 18 | 15 | 15 | 18 | 12 | 15 | 15 |
| 28 | 15 | 18 | 20 | 15 | 18 | 18 | 12 | 15 | 18 |
| 30 | 15 | 18 | 20 | 15 | 18 | 20 | 12 | 15 | 18 |

The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists divided by six feet when reinforced concrete floor is used.
The outside beams to be the same as the intermediate beams.

## TABLE IV.

DEPTH IN INCHES OF I-BEAMS FOR DIFFERENT SPACINGS AND SPANS REQUIRED TO CARRY 20-TON, 15-TON AND 10-TON AUTO TRUCKS AND 30 PER CENT IMPACT. MINIMUM WEIGHTS OF I-BEAMS ARE USED.

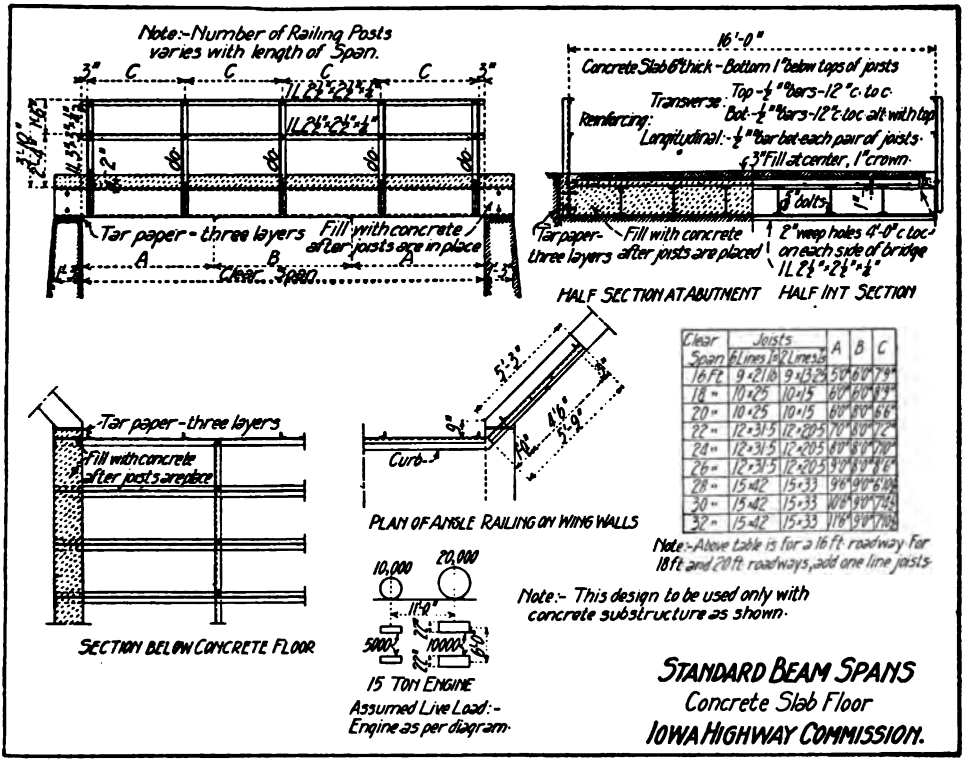
| Plank Floor. | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Span, Ft. | 20-Ton Auto Truck. | | | 15-Ton Auto Truck. | | | 10-Ton Auto Truck. | | |
| | Spacing, Ft. | | | Spacing, Ft. | | | Spacing, Ft. | | |
| | 1½ | 2 | 2½ | 1½ | 2 | 2½ | 1½ | 2 | 2½ |
| 10 | 8 | 9 | 10 | 7 | 8 | 9 | 6 | 7 | 7 |
| 12 | 9 | 10 | 10 | 8 | 9 | 9 | 7 | 7 | 8 |
| 14 | 9 | 10 | 12 | 8 | 9 | 10 | 7 | 8 | 9 |
| 16 | 10 | 12 | 12 | 9 | 10 | 12 | 8 | 8 | 9 |
| 18 | 10 | 12 | 15 | 9 | 10 | 12 | 8 | 9 | 10 |
| 20 | 12 | 12 | 15 | 10 | 12 | 12 | 9 | 9 | 10 |
| 22 | 12 | 15 | 15 | 10 | 12 | 15 | 9 | 10 | 12 |
| 24 | 12 | 15 | 15 | 12 | 12 | 15 | 9 | 10 | 12 |
| 26 | 15 | 15 | 18 | 12 | 15 | 15 | 10 | 12 | 12 |
| 28 | 15 | 15 | 18 | 12 | 15 | 15 | 12 | 12 | 15 |
| 30 | 15 | 18 | 18 | 12 | 15 | 18 | 12 | 12 | 15 |

The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists divided by four feet when timber floor is used.
The outside beams to be the same as the intermediate beams.

Details of standard beam spans designed by the Michigan State Highway Department are given in Fig. 6. The reinforced concrete floor has a bituminous wearing surface. Data and details are given in the cut.

The details of a beam bridge with a concrete floor has designed to comply with the author's specifications are given in Fig. 7.

**Leg Bridges.**—Beam and truss bridges are sometimes supported on steel legs in the place of abutments. A steel leg beam bridge is shown in Fig. 7, Chapter VIII. The legs are composed of I-beams supported on a timber or a steel channel mudsill. The backing is steel plate, or plank,

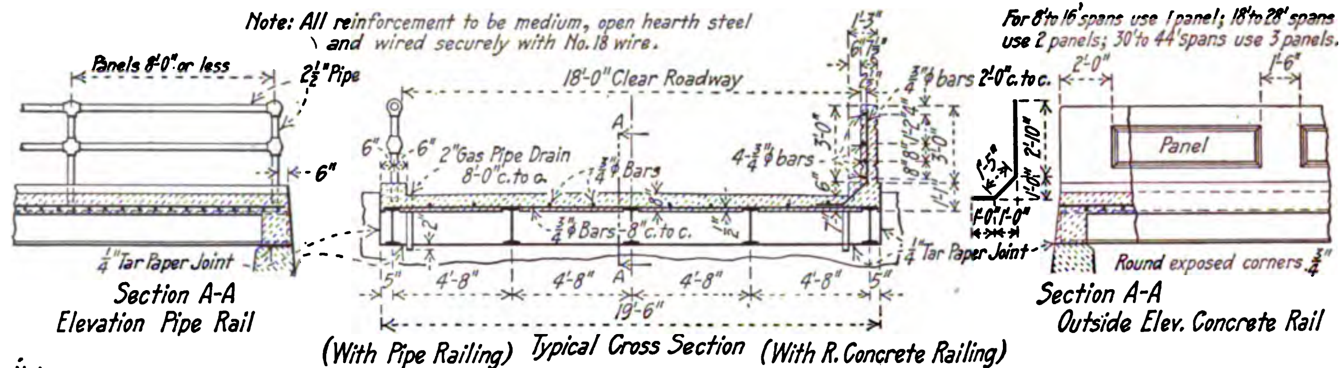



| Clear Span | Joists, 6 Lines I's | Joists, 2 Lines I's | A | B | C |
|---|---|---|---|---|---|
| 16 Ft | 9×21 lb | 9×13.25 | 5'0" | 6'0" | 7'9" |
| 18 " | 10×25 | 10×15 | 6'0" | 6'0" | 8'9" |
| 20 " | 10×25 | 10×15 | 6'0" | 8'0" | 6'6" |
| 22 " | 12×31.5 | 12×20.5 | 7'0" | 8'0" | 7'2" |
| 24 " | 12×31.5 | 12×20.5 | 8'0" | 8'0" | 7'10" |
| 26 " | 12×31.5 | 12×20.5 | 9'0" | 8'0" | 8'6" |
| 28 " | 15×42 | 15×33 | 9'6" | 9'0" | 6'10½" |
| 30 " | 15×42 | 15×33 | 10'6" | 9'0" | 7'4½" |
| 32 " | 15×42 | 15×33 | 11'6" | 9'0" | 7'10½" |

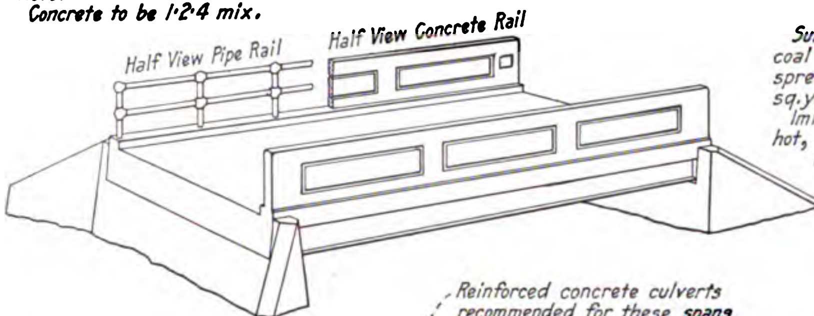
Note:-Above table is for a 16 ft roadway. For 18 ft and 20 ft roadways, add one line joists.

FIG. 5. STANDARD BEAM BRIDGE. IOWA HIGHWAY COMMISSION.

or stone. The legs should be designed to carry the thrust of the filling in addition to the live and dead load on one-half of the span. (For methods of calculation of thrust due to the earth filling, see Chapter XIX, and also the author's "The Design of Walls, Bins and Grain Elevators.") Truss leg bridges should be built with stiff lower chords designed to take the thrust due to the filling. Leg bridges, unless very carefully designed and constructed, are not to be recommended.




Note:
Concrete to be 1:2:4 mix.




SURFACE TREATMENT OF CONCRETE ROADWAY

Surface of the concrete shall be thoroughly cleaned and coal tar, heated to from 250° to 350° Fahr., shall be evenly spread over the surface using not less than ⅓ gallon per sq. yd. of surface.

Immediately after application of tar, and while tar is still hot, the surface shall be evenly covered with ½ inch of clean, coarse, sharp sand.

The application of the tar must not be made when the surface of the concrete is damp.

STANDARD CONCRETE FLOOR FOR I-BEAM SPANS
18'-0" Clear Roadway
Michigan State Highway Department

Reinforced concrete culverts recommended for these spans (8'-0" to 16'-0")

BILL OF MATERIAL

| | Length of Beams | 8'-0" | 10'-0" | 12'-0" | 14'-0" | 16'-0" | 18'-0" | 20'-0" | 22'-0" | 24'-0" | 26'-0" | 28'-0" | 30'-0" | 32'-0" | 34'-0" | 36'-0 | 38'-0" | 40'-0" | 44'-0" |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Floor | Size of Standard Beams | 9"x21# | 10"x25# | 10"x25# | 12"x31.5# | 12"x31.5# | 15"x42# | 15"x42# | 18"x55# | 18"x55# | 18"x55# | 20"x65# | 20"x65# | 24"x80# | 24"x80# | 24"x80# | 24"x80# | 24"x80# | 24"x80# |
| | Weight of Bars in lb. | 385 | 495 | 606 | 717 | 828 | 939 | 1049 | 1159 | 1270 | 1380 | 1491 | 1601 | 1712 | 1822 | 1934 | 2035 | 2156 | 2376 |
| | Vol. of Concrete in cu. yd. | 5.1 | 6.1 | 7.2 | 8.5 | 9.5 | 10.8 | 11.8 | 12.8 | 14.3 | 15.3 | 16.0 | 17.7 | 18.7 | 20.6 | 21.6 | 22.6 | 23.6 | 25.6 |
| Rails | Weight of Bars in lb. | 153 | 193 | 233 | 273 | 313 | 353 | 393 | 433 | 473 | 513 | 553 | 593 | 633 | 673 | 713 | 753 | 793 | 873 |
| | Vol. of Concrete in cu. yd. | 1.1 | 1.4 | 1.7 | 1.9 | 2.2 | 2.5 | 2.8 | 3.1 | 3.3 | 3.6 | 3.9 | 4.2 | 4.4 | 4.7 | 5.0 | 5.3 | 5.6 | 6.1 |

FIG 6. STANDARD BEAM BRIDGE. MICHIGAN STATE HIGHWAY DEPARTMENT.

### Design of a 25 ft. Span I-Beam Bridge with Concrete Floor.

1. **General Description.**—This bridge is to consist of a concrete slab reinforced on the lower side only, resting on I-beams the ends of which bear on channel wall plates. A wearing surface weighing 30 lb. per sq. ft. will be provided.

2. **LOADS.—Dead Load.**—The dead load consists of the weight of the reinforced concrete slab at 150 lb. per cu. ft., the weight of the wearing surface, and the weight of the beams.

**Live Load.**—This bridge will be designed for Class $D_1$ loading, given in the specifications, which provides for a 20-ton concentrated load on two axles, 12 feet apart with wheels spaced 6 feet on the axles, or a uniform live load of 125 lb. per sq. ft. of roadway.

**Impact.**—An allowance of 30 per cent of the live load will be made for impact on the slab and beams.

3. **Dimensions.**—Span, 25 ft. 0 in. c. to c. of end supports. Width of roadway, 16 ft. 0 in. Spacing of beams, 7 spaces at 2 ft. 3 in.

4. **Design of Slab.**—The concentrated load of 20 tons will determine the slab. The width of the rear wheels is $c = 20$ in. or 1.67 ft. In determining the bending moment the effective width of the slab is assumed to be

$$e = \tfrac{2}{3}(l + c) = \tfrac{2}{3}(2.25 + 1.67) = 2.62 \text{ ft.}$$

where $e$ = effective width of distribution of load in feet, $l$ = spacing of beam in feet. The wheel load per foot width of slab considering 30 per cent impact is,

$$P = 1.30 \times 14{,}000 \div 2.62 = 7{,}000 \text{ lb.}$$

The slab will be reinforced on the under side only so must be considered as simply supported. The bending moment, per foot width of slab, due to the wheel load and including impact is

$$M_L = \tfrac{1}{2} \times 7{,}000 \times \tfrac{1}{2} \times 2.25 - \tfrac{1}{2} \times 7{,}000 \times \tfrac{1}{2} \times 0.83 = 2{,}485 \text{ ft.-lb.}$$

Assuming a 6 in. slab and a wearing surface of 30 lb. per sq. ft. the dead load bending moment per foot of width is

$$M_D = \tfrac{1}{8}w \cdot l^2 = \tfrac{1}{8}(75 + 30) \times 2.25^2 = 67 \text{ ft.-lb.}$$

The total bending moment in the slab per foot of width is

$$M = 2{,}485 + 67 = 2{,}552 \text{ ft.-lb.} = 30{,}630 \text{ in.-lb.}$$

For the unit stresses 650 lb. per sq. in. in concrete and 16,000 lb. per sq. in. in steel, required depth to the center of the steel is (see Chapter XVIII).

$$d = \sqrt{\frac{M}{107.5b}} = 0.0965\sqrt{\frac{30{,}630}{12}} = 4.85 \text{ in.}$$

where $M$ = bending moment in inch-pounds.
$d$ = depth from compressive face to center of steel in inches.
$b$ = width in inches over which $M$ is distributed.

A slab with a total thickness of 6 in. and a depth to the center of the steel of $d = 5.0$ in. will be used. From Table I, Chapter X, by interpolation the slab should be $5\tfrac{1}{2}$ in., which checks the calculations. The area of steel per foot of width required to develop this slab is

$$A = 0.0077b \cdot d = 0.0077 \times 12 \times 5.0 = 0.46 \text{ sq. in.}$$

for the unit stresses given in the specifications.

Bars $\tfrac{1}{2}$ in. square and spaced 6 in. c. to c. will be used in the direction perpendicular to the beams. This provides an area of 0.50 sq. in. per foot of width. Two bars $\tfrac{1}{2}$ in. square will be

used between adjacent beams in a direction parallel to the beam to provide for temperature changes and to assist in the distribution of the loads.

In calculating the shear in the slab the wheel load will be considered as distributed over a distance of 3 ft. parallel to the joists. The shear per foot of width is then

$$V = \frac{1.30 \times 14{,}000}{3} \times \frac{1.38}{2.25} + \frac{105 \times 2.25}{2} = 3{,}950 \text{ lb.}$$

The maximum unit shear is

$$f_v = \frac{V}{b \cdot j \cdot d} = 1.15 \frac{V}{b \cdot d} = 1.15 \frac{3{,}950}{12 \times 5.0} = 76 \text{ lb. per sq. in.}$$

The shear is punching shear and it is not necessary to calculate the bond stress.

5. **DESIGN OF BEAMS.**—The beams will be spaced 2 ft. 3 in. as given in § 3 and § 4, and the beam at the edges of the roadway will be the same size as the intermediate beams.

The portion of the wheel load carried by one joist is equal to the spacing of joist in feet divided by six feet, or $2.25 \div 6 = 0.375$.

The load carried by one beam under the rear wheels, and including 30 per cent impact is

$$P = 0.375 \times 1.30 \times 14{,}000 = 6{,}830 \text{ lb.}$$

and under the front wheels

$$P' = 0.375 \times 1.30 \times 6{,}000 = 2{,}925 \text{ lb.}$$

The position of the wheels on the beam for maximum moment is shown in Fig. 7, the center of gravity of the two wheels being as far from one end of the beam as the large wheel is from the other.

$$a = \frac{P' \cdot c}{P' + P} = \frac{2{,}925 \times 12}{2{,}925 + 6{,}830} = 3.60 \text{ ft.}$$

$$b = \frac{l - a}{2} = \frac{25 - 3.6}{2} = 10.7 \text{ ft.}$$

$$M = \frac{(P' + P)b^2}{l} = \frac{9{,}755 \times 10.7^2}{25} = 44{,}675 \text{ ft.-lb.} = 536{,}100 \text{ in.-lb.}$$

The bending moment due to the uniform live load of 125 lb. per sq. ft. of roadway and including 30 per cent impact is

$$M_L = \tfrac{1}{8} \times 1.30 \times (125 \times 2.25) \times 25^2 = 28{,}660 \text{ ft.-lb.}$$

which is less than that due to the concentrated load so need not be considered.

The bending moment due to the weight of the slab and wearing surface is

$$M_D = \tfrac{1}{8} w \cdot l^2 = \tfrac{1}{8}(105 \times 2.25) \times 25^2 = 18{,}450 \text{ ft.-lb.} = 221{,}400 \text{ in.-lb.}$$

The bending moment due to the weight of the beam, which is assumed to weigh 42 lb. per ft., is

$$M_D' = w \cdot l^2/8 = \tfrac{1}{8} \times 42 \times 25^2 = 3{,}290 \text{ ft.-lb.} = 39{,}400 \text{ in.-lb.}$$

The total bending moment in the beam is

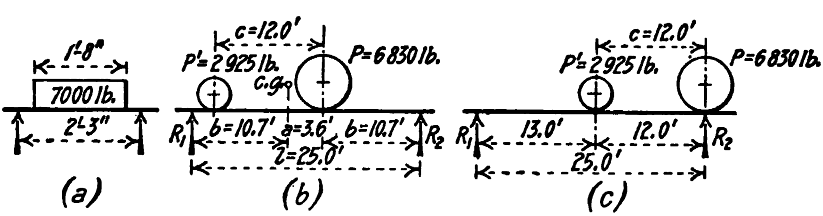



FIG. 7.

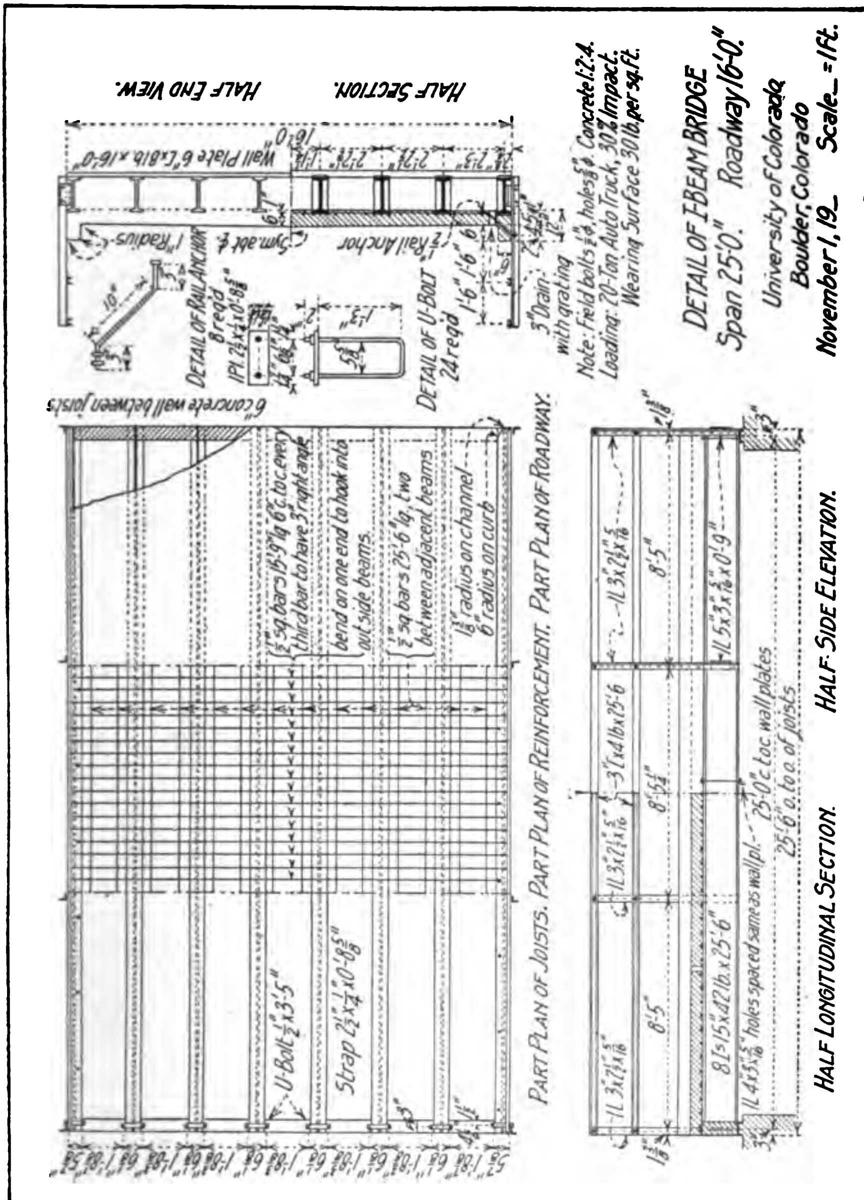

HALF END VIEW.
HALF SECTION.
DETAIL OF RAIL ANCHOR
DETAIL OF U-BOLT
PART PLAN OF JOISTS. PART PLAN OF REINFORCEMENT. PART PLAN OF ROADWAY.
DETAIL OF I-BEAM BRIDGE
Span 25'0". Roadway 16'0".
University of Colorado
Boulder, Colorado
November 1, 19_ Scale _=1ft.
HALF LONGITUDINAL SECTION. HALF SIDE ELEVATION.

FIG. 8.

$$M = 536{,}100 + 221{,}400 + 39{,}400 = 796{,}900 \text{ in.-lb.}$$

The required section modulus is

$$S = \frac{M}{f} = \frac{796{,}900}{16{,}000} = 49.8 \text{ in}^3.$$

A 15 in. I @ 42 lb. provides a section modulus of 58.9 in.$^3$ and will be used. The ratio of depth to span is $15 \div (25 \times 12) = 1/20$. Specifications permit a ratio of 1/30.

The maximum shear due to the 20 ton concentrated load occurs when the wheels are located as shown in (c) Fig. 7 and is

$$V_L = 6{,}830 + \frac{2{,}925 \times 13}{25} = 8{,}350 \text{ lb.}$$

The maximum shear due to the uniform load of 125 lb. per sq. ft. of roadway and including impact is

$$V_L = \tfrac{1}{2}w\cdot l = \tfrac{1}{2}(1.30 \times 125 \times 2.25)25 = 4{,}580 \text{ lb.}$$

which is less than that due to the concentrated load so need not be considered.
The shear due to dead load is

$$V_L = \tfrac{1}{2}w\cdot l = \tfrac{1}{2}(105 \times 2.25 + 42)25 = 3{,}480 \text{ lb.}$$

The total shear on a beam is then

$$V = 8{,}350 + 3{,}480 = 11{,}830 \text{ lb.}$$

The area of the web of a 15 in. I @ 42 lb. is $15 \times 0.41 = 6.15$ sq. in., and the actual unit shear on the gross area of the web is

$$11{,}830 \div 6.15 = 1{,}940 \text{ lb. per sq. in.}$$

The allowable shear is 10,000 lb. per sq. in.

6. **Detail Drawings.**—The detail drawings of this bridge are shown in Fig. 8.

# CHAPTER XII.

## Design of Plate Girder Highway Bridges.

**Introduction.**—A plate girder consists of a vertical steel or iron web plate to whose top and bottom edges are riveted horizontal pairs of angles to form flanges, and to whose ends are attached vertical angles which transmit the load to the supports. Where the web plate is thin as compared with its depth, stiffener angles are riveted on opposite sides of the web, usually in pairs, at intervals not greater than the depth of the girder, or five feet. Where the span is long, two or more plates are spliced together to form the web plates and horizontal plates are riveted to the flange angles to increase the flange area.

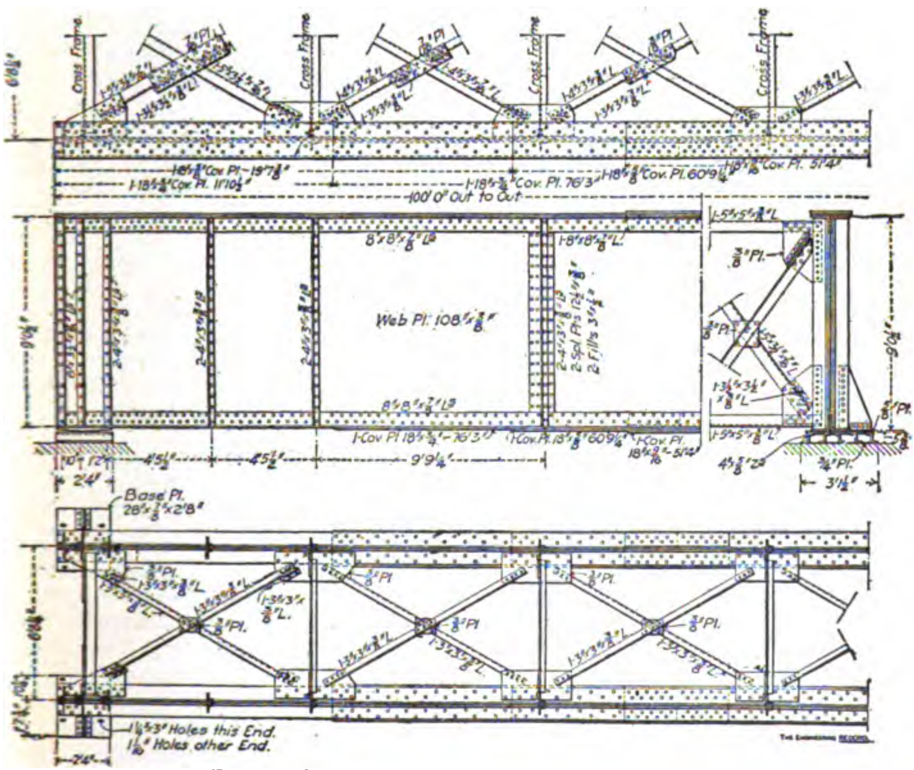



Fig. 1. Railway Deck Plate Girder Bridge.

A plate girder bridge consists of two or more, usually two, plate girders fastened together by lateral bracing, and in the case of deck bridges by transverse bracing consisting of two or more cross-frames. In the railway deck plate girder bridge, Fig. 1, the roadway is carried directly on the tops of the girders. In a through plate girder bridge the roadway is carried on a floor system supported near the bottoms of the plate girders. A through plate girder railroad bridge is shown

in Fig. 2, while a through plate girder highway bridge is shown in Fig. 3. As a through plate girder bridge can have only a lower lateral system, the upper flanges are braced by side braces with gusset plate connections.

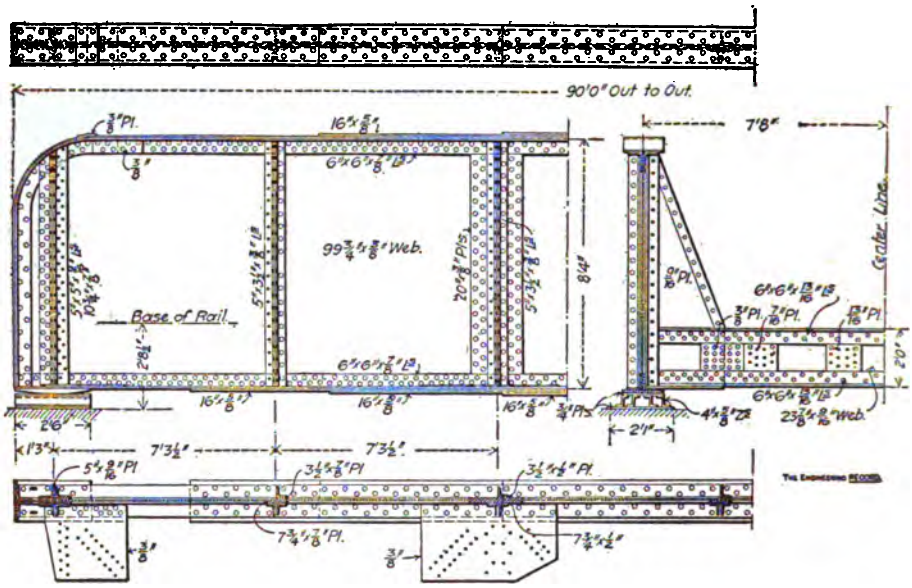



FIG. 2. RAILWAY THROUGH PLATE GIRDER BRIDGE.

Short spans up to 70 or 80 feet have one end fixed while the other end is allowed to move on a sliding plate. For greater lengths of span the expansion end is supported on nests of rollers.

The ordinary limit of plate girder spans is about 100 feet, although railroad plate girders having a span of 126 feet have been built. Plate girders of more than 100 feet span have a depth that makes transportation by rail very difficult.

**Thickness of Web.**—Standard specifications limit the minimum thickness of the web plates to $\frac{3}{8}$ inch for railroad bridges and $\frac{5}{16}$ inch for highway bridges. For heavy loads and long spans the web plates are made much thicker than the minimum thickness. Thin webs require more stiffeners and give a much shorter life to the bridge.

**Flanges.**—The simplest form of a flange consists of a pair of unequal-legged angles with the long legs placed out and riveted to the web plate. When additional rivets are required in the connection of the flanges to the web plate, equal-legged angles with two rows of rivets are used. When additional area is required, one or more cover plates are usually riveted to the horizontal legs of the angles. The thickness of the flanges should be limited so that the rivets will not be longer than five times the diameter of the rivet. Flange angles should never be thinner than the web plates to which they are fastened. Where more than one plate is used, one plate should extend the full length of the girder, the others being continued a short distance (not less than one foot) beyond the point where the area is required. For steam or electric railway plate girder bridges the rivet heads and the variation in the thickness of the flange plates makes it necessary to notch the cross-ties unequally, so that other forms of flange are sometimes used for the upper flanges of long girders. It is quite the common practice to design the tension, or bottom, flange to take the stresses and then make the compression, or upper, flange with the same gross area.

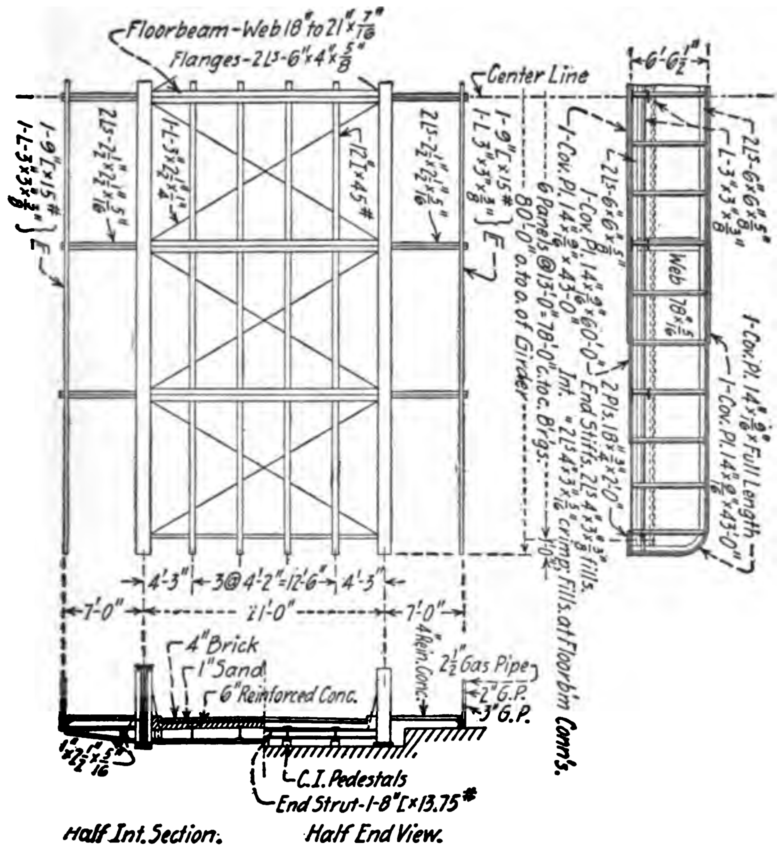



FIG. 3. HIGHWAY THROUGH PLATE GIRDER BRIDGE WITH SOLID FLOOR. (AMERICAN BRIDGE COMPANY.)

**Moments and Shears.**—The moments and the shears in through plate girder bridges, as in Fig. 2 and Fig. 3, are found in the panels in the same manner as for a truss. In a deck bridge the moments and the shears are calculated in a similar manner, at intervals. The load on the girder produces shearing stresses in the girder, which in turn develop tensile and compressive stresses. In a solid rolled beam the entire section carries both shear and bending moment. In plate girders it is usual to assume that all the shear is carried by the web and that all the bending moment is taken by the flanges.

*Nomenclature.*—The following nomenclature will be used.

$M$ = resisting moment of section.
$V$ = vertical shear at section.
$f$ = allowable unit fiber stress.
$I$ = moment of inertia of gross section.
$I'$ = moment of inertia of net section.
$I_w$ = moment of inertia of gross section of web plate.
$I_w'$ = moment of inertia of net section of web plate.
$A_F$ = gross area of one flange.

$A_F'$ = net area of tension flange.
$A_w$ = gross area of web.
$h$ = distance between centers of gravity of flanges.
$h'$ = distance between gage lines of rivets in tension and compression flanges.
$d$ = distance back to back of angles in flanges.
$c$ = distance from neutral axis to extreme fiber.
$p$ = pitch of rivets in flanges.
$r$ = allowable resistance of one rivet.
$w$ = concentrated load per unit length of rail = $P/l$ where $P$ = concentrated load and $l$ = distance over which the load, $P$, is considered as distributed.
$2n$ = number of rivets on one side of web splice.

*Resisting Moment.*—There are four methods now in use for determining the resisting moment of a plate girder section.

(1) Assuming that all the bending moment is carried by the flanges,

$$M = A_F' \cdot f \cdot h \tag{1}$$

(2) Assuming that one-eighth the gross area of the web is available as flange area.

Then the total resisting moment of the girder is

$$M = A_F \cdot f \cdot h + f \cdot t \cdot h^2/6 \tag{2}$$

$$= A_F \cdot f \cdot h + A_w \cdot f \cdot h/6 \tag{3}$$

$$= (A_F + A_w/6) f \cdot h \tag{4}$$

This shows that approximately one-sixth of the web is available as flange area. On account of the reduction of the area due to rivet holes, one-eighth of the area of the web is commonly taken as available as flange area wherever this method is used.

$$M = (A_F' + \tfrac{1}{8} A_w) \cdot f \cdot h \tag{5}$$

(3) By moment of inertia of net section,

$$M = \frac{f \cdot I'}{c} \tag{6}$$

(4) By moment of inertia of gross section (used by American Bridge Co. for plate girders for buildings),

$$M = \frac{f \cdot I}{c} \tag{7}$$

*Rivets in Flanges Which do not Carry Concentrated Loads.*

(1) Assuming that all bending moment is carried by flanges.

The loads produce shearing stresses in the web, which are transferred to the flanges by means of rivets in the flanges. In (c) Fig. 4 let $p$ be the pitch of the flange rivets, $V$ be the vertical shear at section, $h'$ be the distance between lines of rivets in compression and tension flange, and $r$ be the allowable resistance of a rivet; then taking moments about the lower right hand rivet,

$$V \cdot p = r \cdot h'$$

$$p = \frac{r \cdot h'}{V} \tag{8}$$

(2) Assuming that one-eighth the gross area of web is available as flange area,

$$p = \frac{A_F' + \tfrac{1}{8} A_w}{A_F'} \times \frac{r \cdot h'}{V} \tag{9}$$

(3) By moment of inertia of net section,

and

$$p = \frac{2r \cdot I'}{V \cdot A_F' \cdot h} \tag{10}$$

(4) By moment of inertia of gross section,

$$p = \frac{2r \cdot I}{V \cdot A_F \cdot h} \qquad (11)$$

*Rivets in Flanges Carrying Concentrated Loads.*

(1) Assuming that all the bending moment is carried by the flanges,

$$p = \frac{r}{\sqrt{w^2 + \left(\frac{V}{h'}\right)^2}} \qquad (12)$$

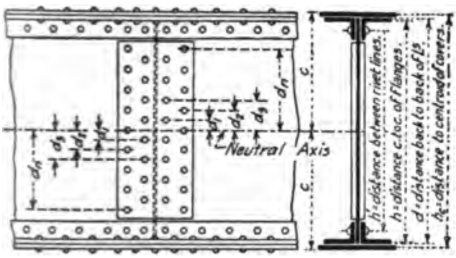



FIG. 4. WEB SPLICE FOR PLATE GIRDER.

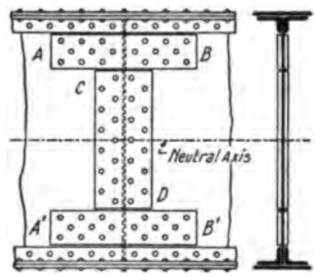



FIG. 5. WEB SPLICE FOR PLATE GIRDER.

(2) Assuming that one-eighth the gross area of the web is available as flange area,

$$p = \frac{r}{\sqrt{w^2 + \left(\frac{A_F'}{A_F' + \frac{1}{8}A_w} \cdot \frac{V}{h}\right)^2}} \qquad (13)$$

(3) By moment of inertia of net section,

$$p = \frac{r}{\sqrt{w^2 + \left(\frac{V \cdot A_F' \cdot h}{2I'}\right)^2}} \qquad (14)$$

(4) By moment of inertia of gross section,

$$p = \frac{r}{\sqrt{w^2 + \left(\frac{V \cdot A_F \cdot h}{2I}\right)^2}} \qquad (15)$$

*Rivets Connecting Cover Plates to Flange Angles.*

(1) and (2). Assuming that all the bending moment is carried by the flanges, or that one-eighth the gross area of the web is available as flange area,

$$p = \frac{2n \cdot r \cdot d \cdot A_F}{V \cdot A_c'} \qquad (16)$$

where $n$ = number of rivets on one transverse line.
$r$ = value of one rivet in single shear or bearing.
$d$ = distance back to back of angles.
$A_c'$ = total net area of cover plates in one flange.

(3) By moment of inertia of net section,

$$p = \frac{2n \cdot I' \cdot r}{V \cdot A_c' \cdot h_c} \qquad (17)$$

where $A_c'$ = total net area of cover plates in one flange.

$h_c$ = distance between centroids of all cover plates in tension flange and all cover plates in compression flange.

(4) By moment of inertia of gross section,

$$p = \frac{2n \cdot I \cdot r}{V \cdot A_c \cdot h_c} \tag{18}$$

where $A_c$ = total gross area of cover plates in one flange.

$h_c$ = distance between centroids of all cover plates in tension flange and all cover plates in compression flange.

*Web Splice.*—An ordinary web splice is shown in Fig. 4. Where splice plates are designed to carry part of the moment as well as the shear the splice shown in Fig. 5 is sometimes used. Plates $AB$ and $A'B'$ are assumed to transfer that part of the moment carried by the web, and plate $CD$ to transfer the shear. Two lines of rivets should be used in each section of the web spliced. The number and spacing of rivets in a web splice can be determined only by trial, except when the first method for proportioning the section is used. The rivet most remote from the neutral axis is the most severely stressed.

(1) Assuming that all the bending moment is carried by the flanges,

$$r = \frac{V}{2n}, \quad \text{and} \quad 2n = \frac{V}{r} \tag{19}$$

(2) Assuming that one-eighth the area of web is available as flange area. The stress in the outermost rivet is given by the formula, where $M'$ is moment carried by web.

$$r = \sqrt{\left(\frac{V}{2n}\right)^2 + \left(\frac{M' \cdot d_n}{2\Sigma d^2}\right)^2} \tag{20}$$

(3) By moment of inertia of net section. The stress in the outermost rivet is given by the formula

$$r = \sqrt{\left(\frac{V}{2n}\right)^2 + \left(\frac{I_w'}{I'} \cdot \frac{M \cdot d_n}{2\Sigma d^2}\right)^2} \tag{21}$$

(4) By moment of inertia of gross section. The stress in the outermost rivet is given by the formula

$$r = \sqrt{\left(\frac{V}{2n}\right)^2 + \left(\frac{I_w}{I} \cdot \frac{M \cdot d_n}{2\Sigma d^2}\right)^2} \tag{22}$$

*Flange Splice.*—Flanges should never be spliced unless it is impossible to get material of the required length. Flange splices should always be located at points where there is an excess of flange section, no two parts of the flange should be spliced within two feet of each other. Rivets in splice plates and angles should be located as close together as possible in order that the transfer may take place in a short distance. No allowance should be made for abutting edges of spliced members of the compression flange.

Flange angles should be spliced with a splice angle of equal section riveted to both legs of the angle spliced. Where this is impossible the largest possible splice angle should be used and the difference made up by a plate riveted to the vertical leg of the opposite angle. The number of rivets required in the splice angle on each side of the joint in the angle is given by the formula

$$n = \frac{f \cdot A}{r} \tag{23}$$

where $f$ = the allowable unit stress in the flange, $A$ = area of spliced angle, and $r$ = the allowable stress on one rivet. Rivets which are already considered as transferring the shear may be considered as splice rivets if they are included in the splice angle.

Cover plates should be spliced with a splice plate of equal section. The number of rivets

required in the splice plate on each side of the joint is determined by the above formula if the plates are in direct contact in the same way as for splice angles. Where one or more plates intervene between the splice plate and cover plate which it splices, rivets should be used on each side of the joint in excess of the number required in case of direct contact, to an extent of one-third that number for each intervening plate.

The above methods for flange splicing apply only when methods (1) and (2) of proportioning sections are used, but may be used with sufficient accuracy when methods (3) and (4) are used. Strictly speaking for methods (3) and (4) splice angles and plates should have moments of inertia about the neutral axis, equal to the moments of inertia of the members they splice, about the neutral axis. An exact analysis for the number of rivets required in splices would give a less number than obtained from above formula.

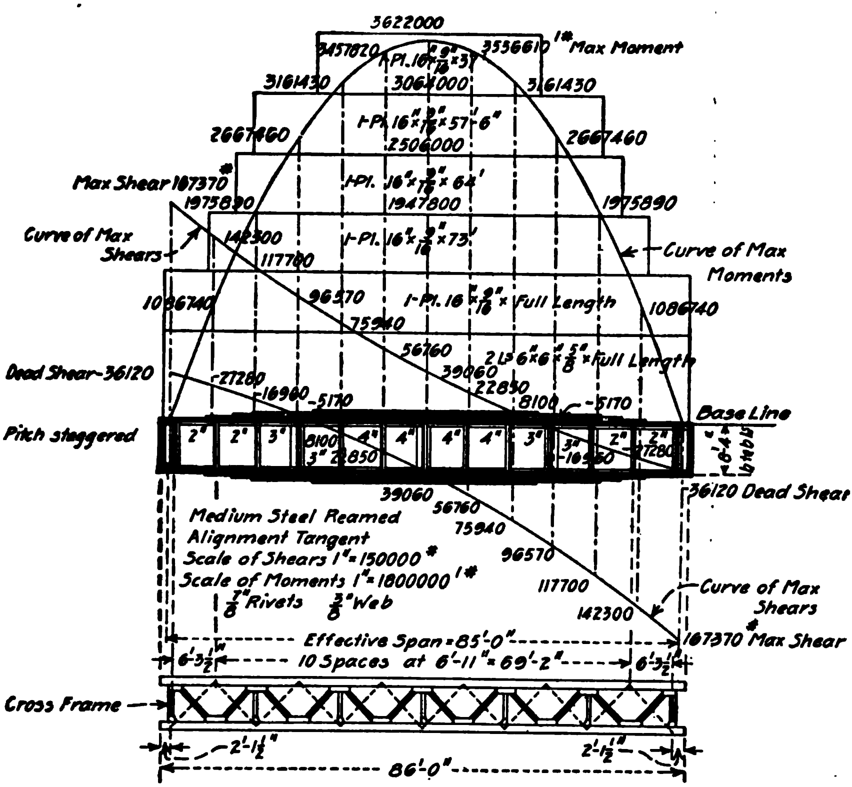



FIG. 6. SHEARS AND MOMENTS IN A RAILWAY PLATE GIRDER.

**Design of Web Stiffeners.**—Web stiffeners are used to prevent the buckling of the web plate, and are usually spaced somewhat less than the depth of the girder. There is no rational method for the design of stiffener angles. Tests show that the stiffener acts as a beam to prevent buckling of the web and is not appreciably stressed in the direction of its length, except where it is used at points of concentrated loading. A common specification is that "The distance between stiffeners shall not exceed that given by the following formula (and not greater than the clear depth of the web); $d = t(12,000 - s)/4$, where $d$ = clear distance between stiffeners of flange angles, $t$ = thickness of web, $s$ = shear lb. per sq. in. Where stiffeners are required they shall

be designed as columns with an allowable unit stress of $P = 16{,}000 - 70\,l/r$, where $l$ = the one-half the depth of girder and $r$ = the radius of gyration of the stiffener angles at right angles to the web plate, both in inches." "Stiffeners shall be provided at ends and at all points of concentrated loading, and shall contain enough rivets to transfer the vertical shear to the web plate."

**Camber.**—Plate girders are cambered by separating the web plates by the required amount, in the upper part of the web splice. Plate girders in which a single web plate is used without a splice cannot be cambered. Many engineers do not camber plate girders.

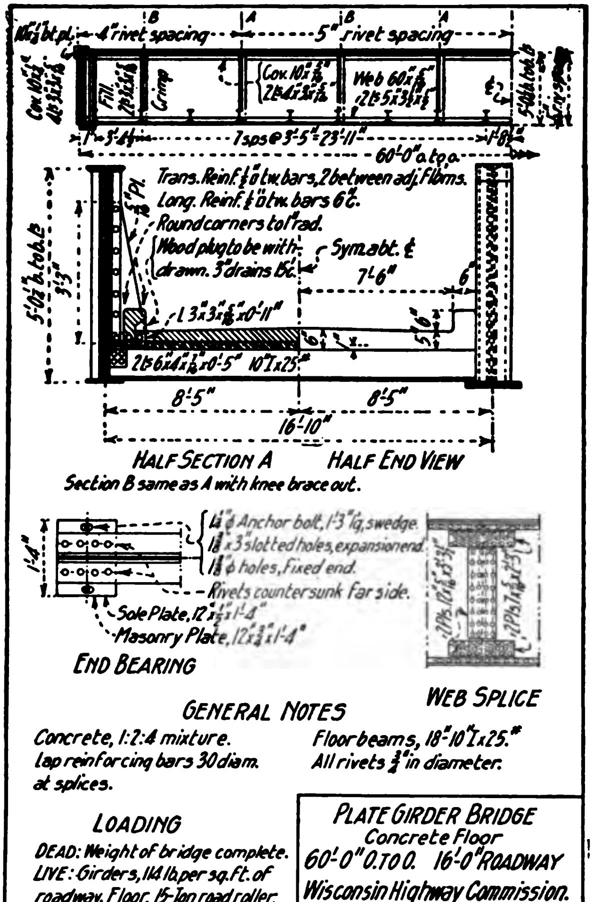



FIG. 7. PLATE GIRDER HIGHWAY BRIDGE. WISCONSIN HIGHWAY COMMISSION.

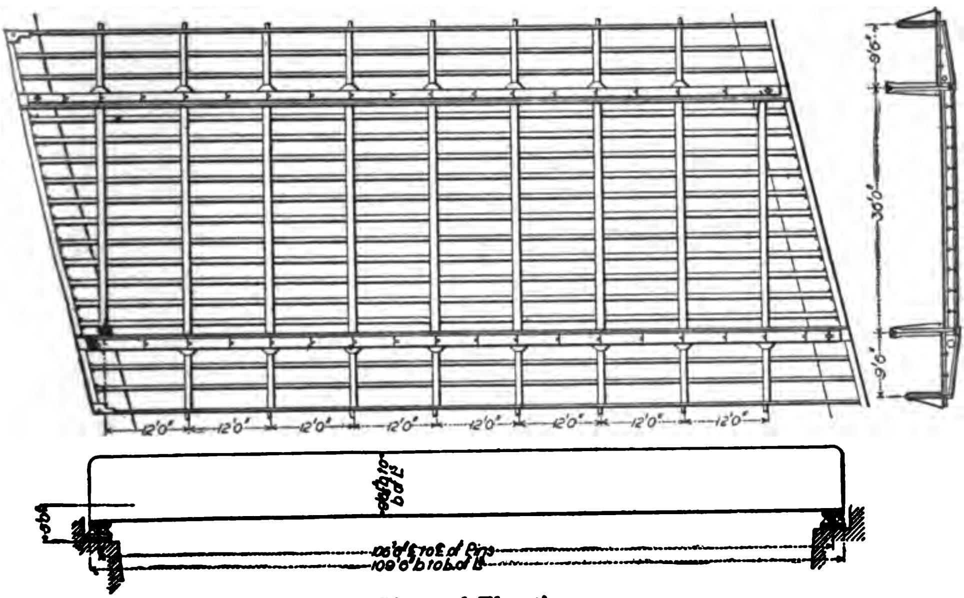
**Economical Depth.**—Plate girders are commonly made with a depth of from $\frac{1}{8}$ to $\frac{1}{12}$ of the span. An approximate empirical rule for the depth is to make the area of the two flanges equal to the area of the web plate.

**Example of Calculation.**—The maximum shears and moments in an 86-foot span deck plate girder railway bridge are shown in Fig. 6. The shear at any point may be found by scaling from the shear curve to the horizontal base line, while the moments may be obtained by scaling from

the moment curve to the base line. The composition of the flanges and the length of the plates are shown. The maximum shears and bending moments were calculated at intervals of about 7 feet, and the curves were drawn through these points.

**Details of Plate Girders.**—The general plans for an 80-foot through plate girder highway bridge are shown in Fig. 3. The roadway is 20 feet in the clear, with two 6½-ft. sidewalks. The floor of the roadway consists of a 6-in. reinforced concrete slab carried on 12-inch I beam joists, covered with a wearing surface of paving brick.

Details of a steel through plate girder highway bridge as designed by the Wisconsin Highway Commission are shown in Fig. 7. Standard plans have been prepared for spans from 35 ft. to 80 ft., varying by 5 ft. intervals, and for 16 ft., 18 ft. and 20 ft. roadway. Data for plate girder highway bridges designed by the Wisconsin Highway Commission are given in Table I.




**Floor Plan and Elevation.**

FIG. 8. DETAILS OF 109-FT. PLATE GIRDER HIGHWAY BRIDGE. (Engineering Record, May 21, 1910.)

Details of a 109-ft. span through plate girder highway bridge built over the D. L. & W. R. R. tracks in Jersey City, N. J., are given in Fig. 8 and Fig. 9. The girders were designed for a live load of 108 lb. per sq. ft. on roadway and sidewalk; while the roadway floor was designed for a live load of 100 lb. per sq. ft. and two 12,000 lb. axle loads spaced 10 ft. apart with an allowance of 25 per cent for impact. The expansion end is carried on 4 in. rollers. The concrete has a minimum thickness of 4 in. and is covered with 1½ in. of binder and 2 in. of asphalt. Each main girder weighed 112,000 lb.; and the total weight of steel in the bridge was about 403,000 lb.

The detailed shop plans of a 65-ft. deck plate girder electric railway bridge are shown in Fig. 10. This girder span was designed according to Cooper's 1901 Specifications for $E_1$ loading. The ties were carried directly on the top flanges of the girders. These details represent good practice in light plate girder construction. It will be noted that the stiffener angles are crimped to go over the flange angles except where two hitch stiffener angles are used for cross-frame connections, in which case fillers are used. The ties were fastened to the upper flange angles by

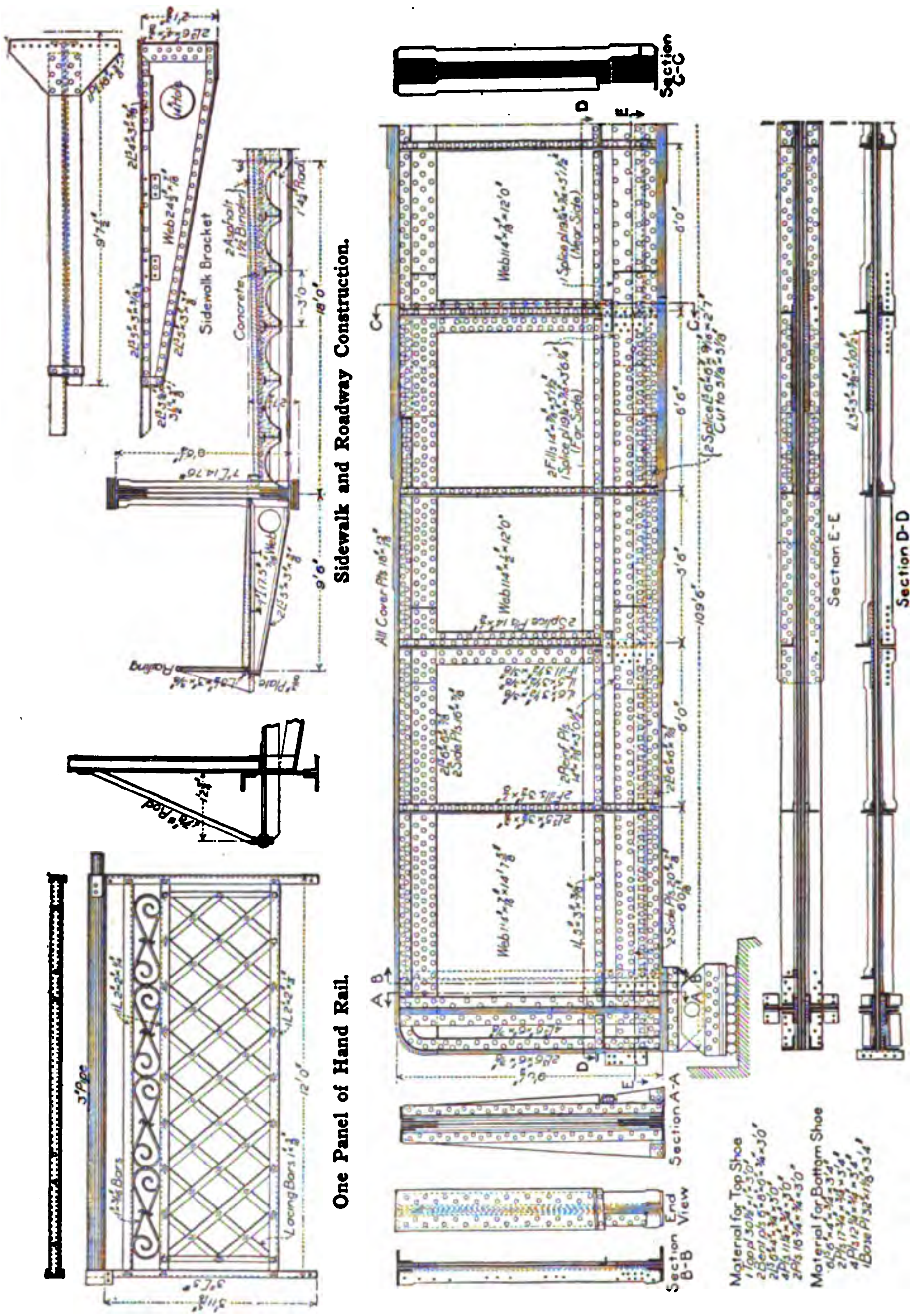
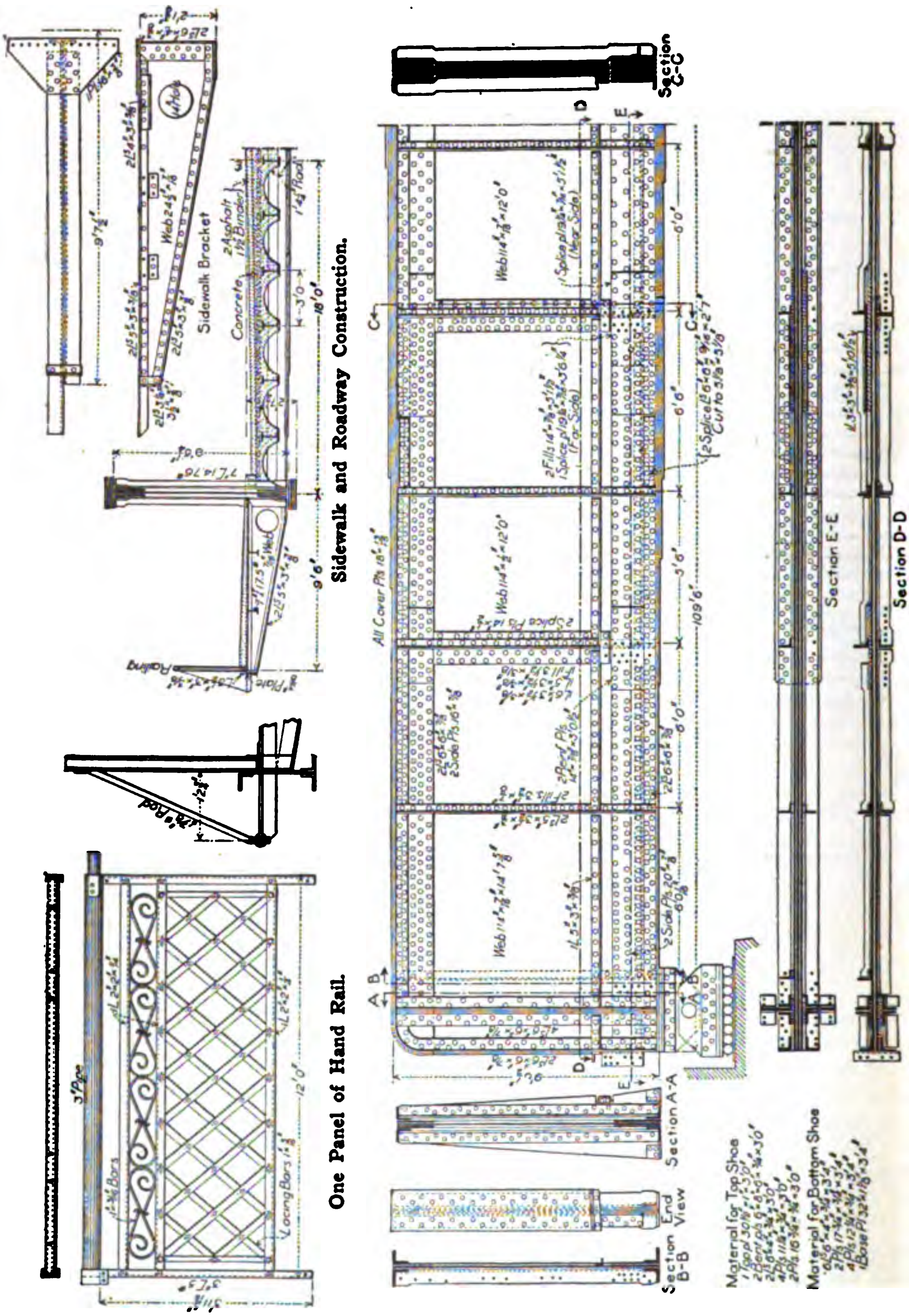
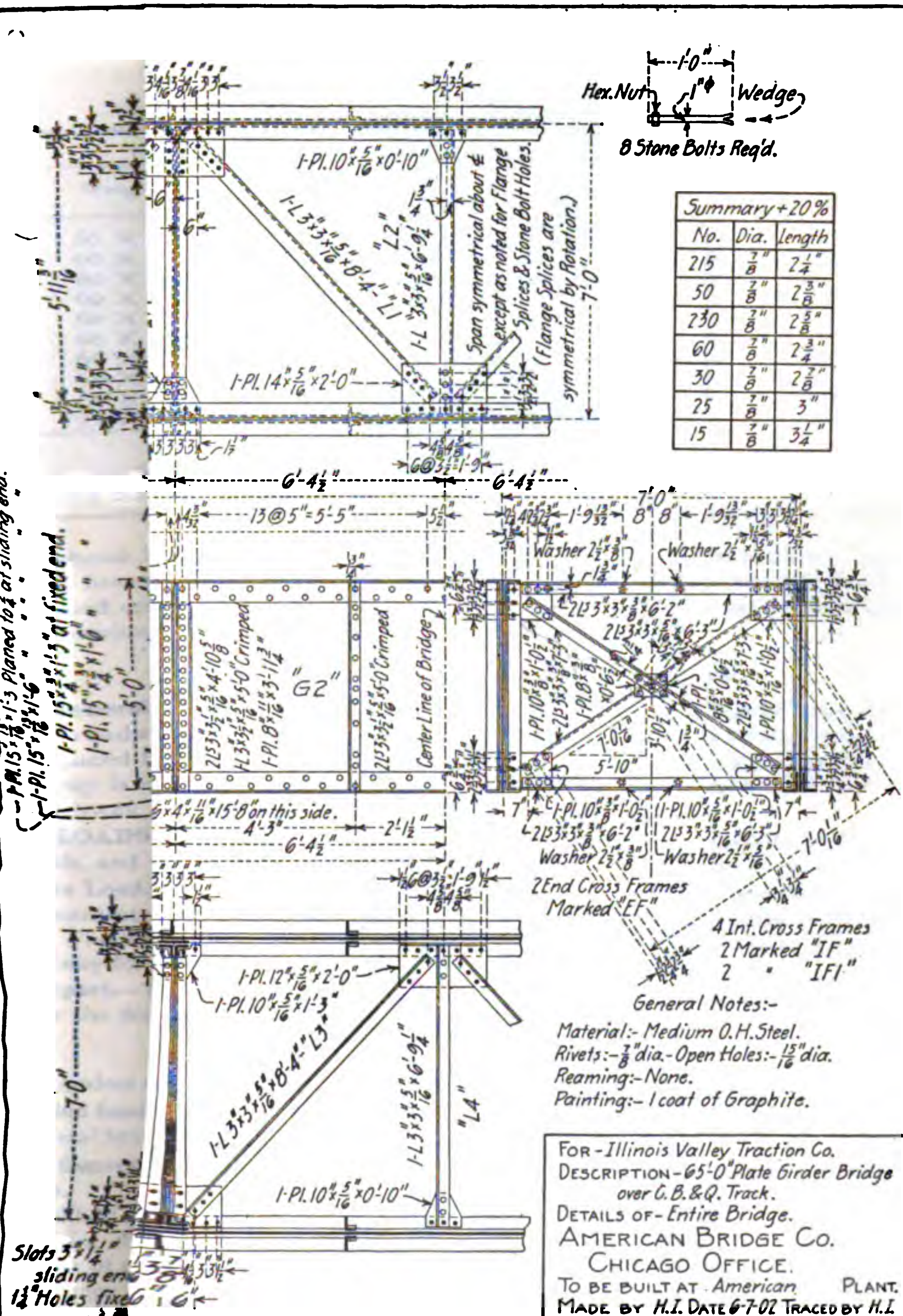



FIG. 9. DETAILS OF 109-FT. PLATE GIRDER HIGHWAY BRIDGE.
(Engineering Record, May 21, 1910.)




Elevation of Girder Showing Flange Splices.

FIG. 9. DETAILS OF 109-FT. PLATE GIRDER HIGHWAY BRIDGE.
(Engineering Record, May 21, 1910.)




| Summary + 20% | | |
|---|---|---|
| No. | Dia. | Length |
| 215 | 7/8" | 2 1/4" |
| 50 | 7/8" | 2 3/8" |
| 230 | 7/8" | 2 5/8" |
| 60 | 7/8" | 2 3/4" |
| 30 | 7/8" | 2 7/8" |
| 25 | 7/8" | 3" |
| 15 | 7/8" | 3 1/4" |

General Notes:-

Material:- Medium O.H. Steel.
Rivets:- 7/8" dia. - Open Holes:- 15/16" dia.
Reaming:- None.
Painting:- 1 coat of Graphite.

FOR - Illinois Valley Traction Co.
DESCRIPTION - 65'-0" Plate Girder Bridge over C.B.&Q. Track.
DETAILS OF - Entire Bridge.
AMERICAN BRIDGE CO.
CHICAGO OFFICE.
TO BE BUILT AT American PLANT.
MADE BY H.I. DATE 6-7-02 TRACED BY H.I.
DATE 6-10-02 CHECKED BY B.W.P. DATE 6-12-02.
CONTRACT No. A7522.

## TABLE I.

### Steel Plate Girder Highway Spans
### Wisconsin Highway Commission.

| Span Ft. | 16 Ft. Roadway. | | 18 Ft. Roadway. | | 20 Ft. Roadway. | |
|---|---|---|---|---|---|---|
| | Web, In. | Weight of Steel, Lb. | Web, In. | Weight of Steel, Lb. | Web, In. | Weight of Steel, Lb. |
| 35 | $60 \times \frac{5}{16}$ | 15,430 | $60 \times \frac{5}{16}$ | 16,150 | $60 \times \frac{5}{16}$ | 18,480 |
| 40 | $60 \times \frac{5}{16}$ | 17,160 | $60 \times \frac{5}{16}$ | 18,110 | $60 \times \frac{5}{16}$ | 21,200 |
| 45 | $60 \times \frac{5}{16}$ | 19,680 | $60 \times \frac{5}{16}$ | 20,940 | $60 \times \frac{5}{16}$ | 23,760 |
| 50 | $60 \times \frac{5}{16}$ | 21,530 | $60 \times \frac{5}{16}$ | 23,400 | $60 \times \frac{5}{16}$ | 27,360 |
| 55 | $60 \times \frac{5}{16}$ | 24,270 | $60 \times \frac{5}{16}$ | 26,800 | $60 \times \frac{5}{16}$ | 30,790 |
| 60 | $60 \times \frac{5}{16}$ | 27,300 | $60 \times \frac{5}{16}$ | 30,310 | $60 \times \frac{5}{16}$ | 35,270 |
| 65 | $66 \times \frac{5}{16}$ | 30,910 | $66 \times \frac{5}{16}$ | 33,390 | $66 \times \frac{5}{16}$ | 38,840 |
| 70 | $66 \times \frac{5}{16}$ | 36,300 | $66 \times \frac{3}{8}$ | 39,680 | $66 \times \frac{3}{8}$ | 45,960 |
| 75 | $66 \times \frac{3}{8}$ | 40,580 | $72 \times \frac{3}{8}$ | 43,960 | $72 \times \frac{3}{8}$ | 50,510 |
| 80 | $72 \times \frac{3}{8}$ | 44,770 | $78 \times \frac{3}{8}$ | 48,360 | $78 \times \frac{5}{8}$ | 56,260 |

| | 16 ft. Roadway. | 18 ft. Roadway. | 20 ft. Roadway. |
|---|---|---|---|
| Concrete, per lineal foot | 0.29 cu. yd. | 0.325 cu. yd. | 0.358 cu. yd. |
| Reinforcing Steel, per lineal foot | 35 lb. | 38 lb. | 45 lb. |

means of hook bolts in every third tie, while the 6 × 8 inch guards were bolted to the ties. The upper and lower laterals are of the Warren type and are made of single angles. Four intermediate and two end cross-frames are used. This bridge was designed by the author and was fabricated by the American Bridge Company.

### Design of a 50-ft. Span Through Plate Girder Bridge.

1. **General Description of Bridge.**—This is to be a 50 ft. through plate girder bridge with an 18 ft. roadway. The floor is to consist of a reinforced concrete floor slab resting directly on closely spaced floorbeams, no stringers being used. A wearing surface weighing 30 lb. per sq. ft. of roadway is used. The bridge will be designed to comply with the General Specifications for Steel Highway Bridges given in Appendix I, to which reference will be made.

2. **LOADS. Dead Load.**—The dead load consists of the weight of the girders, floorbeams, floor slab, and wearing surface. Reinforced concrete is assumed to weigh 150 lb. per cu. ft.

**Live Load.**—This bridge will be designed for Class $D_2$ loading which provides for a 15-ton concentrated load or a uniform load of 100 lb. per sq. ft. of roadway for the floor and its supports. The live load for the girders is given in Table I of the specifications as 90 lb. per sq. ft. of roadway for a span of 50 feet.

**Impact.**—The specifications provide for an allowance for impact of 30 per cent of the live load for the floor and its supports and of

$$100/(L + 300) = 100/349 = 28.7 \text{ per cent}$$

for the girders assuming distance center to center of bearings as 49 ft.

**Wind Load.**—The wind load will have no appreciable effect on a plate girder span of this type. The lateral bracing is omitted on account of the rigidity of the concrete floor.

3. **General Dimensions.**—Span 50′ 0″ out to out or about 49′ 0″ center to center of end bearings.

Width of roadway, 18′ 0″.

Spacing of girders, 18′ 10″ about, center to center.

Depth of girder (§53) must be at least $\frac{1}{12} \times 50 \times 12 = 50$ in. A depth of $54\frac{1}{2}$ in. back to back of flange angles will be used.

Rivets $\frac{3}{4}$ in. in diameter will be used throughout the bridge.

4. **Detailed Dimensions.**—Gusset plates will be used to provide lateral support for the girders (§ 6). These plates must come at the floorbeams and should be connected to the stiffeners if stiffeners are used so that additional connections will not have to be provided. If the thickness of the web plate is less than $\frac{1}{60}$ of its unsupported width, or $\frac{1}{60}\times$ about $50 = 0.83$ in. stiffeners must be used (§ 50). The thickness of the web plate, if stiffeners are used can be as small as $50 \div 160 = \frac{5}{16}$ in. This is the minimum thickness which should be used for highway bridges and will be adopted for this short span, so stiffeners will be required. The spacing of stiffeners should not exceed the clear depth of the web or about 50 in. (§ 51). If stiffeners are placed at every floorbeam the spacing of floorbeams would be 50 inches or less, and at alternate floorbeams 25 inches or less. Calculation should be made to determine the most economical arrangement. A spacing of 4 feet with stiffeners at every floorbeam was found to be the most economical for this case. Gusset plates will be placed at alternate floorbeams or 8′ 0″ apart.

5. **DESIGN OF SLAB.**—The concentrated load of 15 tons will determine the slab. In calculating the bending moment in the slab a wheel load will be considered as distributed on a line parallel to the floorbeam and having a length as given by the formula (38), Chapter IX.

$$e = 2l/3 + c$$

with a maximum of 6 ft.

where $e$ = effective width of distribution of load in feet,

$l$ = spacing of floorbeams in feet,

$c$ = width of wheel in feet = 15 in. or 1.25 ft.

For $l = 4.0$ ft., $e = \frac{2}{3} \times 4 + 1.25 = 3.92$ ft. The wheel load per foot of width of slab, including 30 per cent impact is

$$P = 1.30 \times 10{,}000 \div 3.92 = 3{,}320 \text{ lb.}$$

The slab will be reinforced on both sides so the intermediate spans may be considered as continuous and the end spans as partially continuous. Since it is undesirable to change the thickness of the slab for the intermediate panels all slabs will be made of the dimensions required for the end spans. Considering the slab to be simply supported, the bending moment due to live load and impact is

$$M_L = \tfrac{1}{4}P\cdot l = \tfrac{1}{4} \times 3{,}320 \times 4.0 = 3{,}320 \text{ ft.-lb. per ft. width.}$$

For a partially continuous slab the negative bending moment at the support and the maximum positive moment are

$$M_L = \tfrac{4}{5} \times 3{,}320 = 2{,}650 \text{ ft.-lb. per ft. width.}$$

Assuming a 6 in. slab with a wearing surface of 30 lb. per sq. ft. the total dead load is $75 + 30 = 105$ lb. per sq. ft., and considering the slab simply supported the bending moment is

$$M_D = \tfrac{1}{8}w\cdot l^2 = \tfrac{1}{8} \times 105 \times 4^2 = 210 \text{ ft.-lb.}$$

and for a partially continuous slab

$$M_D = \tfrac{4}{5} \times 210 = 170 \text{ ft.-lb.}$$

The total bending moment per foot width of slab is

$$M = 2{,}650 + 170 = 2{,}820 \text{ ft.-lb.} = 33{,}800 \text{ in.-lb.}$$

For the unit stresses given in the specifications the required depth to the center of the steel from formula (6c), Chapter XVIII is

$$d = 0.0965\sqrt{\frac{M}{b}} = 0.0965\sqrt{\frac{33{,}800}{12}} = 5.12 \text{ in.}$$

where $M$ = bending moment in inch-pounds.

$d$ = depth from compressive face to center of steel.

$b$ = width in inches over which $M$ is distributed.

A slab with a total thickness of 6.25 in. and a depth to the center of the steel of $d = 5.25$ in. will be used. The area of steel per foot width, required to develop this slab is

$$A = 0.0077 b \cdot d = 0.0077 \times 12 \times 5.25 = 0.49 \text{ sq. in.}$$

for the unit stresses given in the specifications. Bars ½ in. square spaced 6 in. c. to c. will be used in the direction perpendicular to the floorbeams at both top and bottom of the slab. These bars provide an area of 0.50 sq. in. per foot of width. Three ½ in. square bars will be used between adjacent floor beams at the bottom of the slab in the direction parallel to the floorbeams and one at each floorbeam at the top of the slab to provide for temperature changes and to assist in the distribution of the load.

In calculating shear and bond stress the maximum end shear is distributed over a width of three feet.

The maximum shear in the slab is

$$V = P + \tfrac{1}{2} w \cdot l = \frac{1.30 \times 10{,}000}{3} + \tfrac{1}{2} \times 108 \times 4 = 4{,}550 \text{ lb.}$$

The maximum unit shear is

$$f_v = \frac{V}{b \cdot j \cdot d} = 1.15 \frac{V}{b \cdot d} = 1.15 \frac{4{,}550}{12 \times 5.25} = 83 \text{ lb. per sq. in.}$$

The shear is punching shear and it is not necessary to calculate bond stress.

The thickness of slab from Table II, Chapter X, is 6.25 in.

6. **DESIGN OF FLOORBEAMS.**—As previously noted the floorbeams will be spaced 4′ 0″ centers, the end floorbeams being of the same section as the intermediate floorbeams. In determining the amount of the axle load carried by one floorbeam the load may be considered as distributed over a distance of 12 ft. parallel to the axle. The percentage of the axle load carried by one floorbeam is equal to the spacing divided by 6 (§ 19). The load carried by one floorbeam and including 30 per cent impact is

$$\tfrac{2}{3} \times 1.30 \times 20{,}000 = 17{,}300 \text{ lb.}$$

The live load bending moment in the floorbeam is, from (a) Fig. 11.

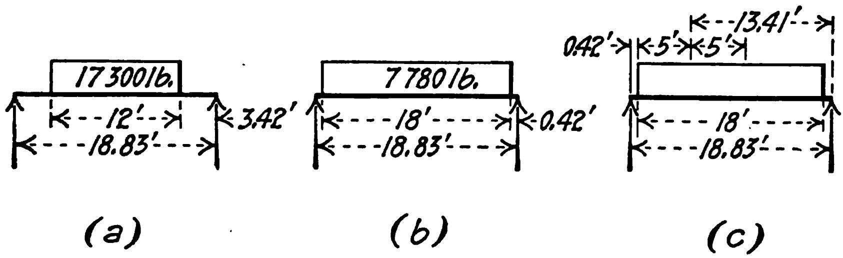



FIG. 11.

$$M_L = \tfrac{1}{2} \times 17{,}300 \times \tfrac{1}{2} \times 18.83 - \tfrac{1}{2} \times 17{,}300 \times \tfrac{1}{2} \times 6 = 55{,}600 \text{ ft.-lb.}$$

The dead load due to the weight of the slab and wearing surface is $4 \times 108 \times 18 = 7{,}780$ lb. distributed over 16 ft. of the floorbeam. The dead load bending moment due to the weight of the slab is, from (*b*) Fig. 11

$$M_D = \tfrac{1}{2} \times 7{,}780 \times \tfrac{1}{2} \times 18.83 - \tfrac{1}{2} \times 7{,}780 \times \tfrac{1}{2} \times 9 = 19{,}100 \text{ ft.-lb.}$$

Assuming the weight of the floorbeam to be 42 lb. per foot, the bending momnt due to this load is

$$M_D' = \tfrac{1}{8} \times 42 \times 18.83^2 = 1{,}860 \text{ ft.-lb.}$$

The total bending moment in the floorbeam is

$$M = 55{,}600 + 19{,}100 + 1{,}860 = 76{,}560 \text{ ft.-lb.} = 919{,}000 \text{ in.-lb.}$$

The required section modulus is

$$S = \frac{M}{f} = \frac{919{,}000}{16{,}000} = 57.4 \text{ in}^3.$$

A 15 in. I (@) 42 lb. provides a section modulus of 58.9 in³. and will be used.

The live load end shear including 30 per cent impact and considering the clearance line of the load as at the edge of the roadway is from (*c*) Fig. 11.

$$V_L = 17{,}300 \times 13.41 \div 18.83 = 12{,}300 \text{ lb.}$$

The dead load end shear is

$$V_D = \tfrac{1}{2} \times 7{,}780 + \tfrac{1}{2} \times 42 \times 18.83 = 4{,}290 \text{ lb.}$$

The total end shear is

$$V = 12{,}300 + 4{,}290 = 16{,}590 \text{ lb.}$$

The unit shear on the web of the floorbeam is $16{,}590 \div 15 \times 0.41 = 2{,}700$ lb. per sq. in. Allowable unit shear = 10,000 lb. per sq. in. (§ 40).

The number of ¾ in. field rivets required between the connection angles of the floorbeam and the web of the girder is $16{,}590 \div 4{,}420 = 4$ rivets, and the number of ¾ in. shop rivets required between the connection angles and the web of the floorbeam is

$$16{,}590 \div (24{,}000 \times 0.75 \times 0.41) = 3 \text{ rivets.}$$

7. **STRESSES IN GIRDERS.**—The girder section will be made uniform throughout the entire length for there is no economy in varying the light section which will be required for this load and span. A cover plate will be used on the top flange for the full length of span to keep out water and to improve the appearance of the girder. No cover plate will be used on the bottom flange unless the bending moment makes it advisable to use that type of section.

All loads will be considered as uniformly distributed in calculating the stresses in the girder, instead of concentrating the floor loads at the floorbeam points.

The total dead load for the bridge is found as follows:

| | | |
|---|---|---|
| 13 Floorbeams, 13 × 42 lb. × 18.8 | = | 10,250 lb. |
| Slab and wearing surface, 105 × 18 × 50 | = | 94,500 " |
| Girders assumed, | | 16,000 " |
| Total dead load for entire span | = | 120,750 lb. |

The dead load bending moment at the center for one girder is

$$M_D = \tfrac{1}{8} W_D \cdot l = \tfrac{1}{8} \times \frac{120{,}750}{2} \times 49 = 370{,}000 \text{ ft.-lb.}$$

The dead load end shear for one girder is

$$V_D = \tfrac{1}{2} W_D = \frac{120{,}750}{2 \times 2} = 30{,}200 \text{ lb.}$$

The live load on the entire span and including 28.7 per cent impact is,

$$1.287 \times 90 \times 18 \times 50 = 104{,}200 \text{ lb.}$$

The live load bending moment at the center for one girder is

$$M_L = \tfrac{1}{8} W_L \cdot l = \tfrac{1}{8} \times \frac{104{,}200}{2} \times 49 = 320{,}000 \text{ ft.-lb.}$$

The live load end shear for one girder is,

$$V_L = \tfrac{1}{2} W_L = \frac{104{,}200}{2 \times 2} = 26{,}100 \text{ lb.}$$

The total bending moment at the center for one girder is

$$M = M_D + M_L = 690{,}000 \text{ ft.-lb.} = 8{,}280{,}000 \text{ in.-lb.}$$

The total end shear for one girder is

$$V = V_D + V_L = 56{,}300 \text{ lb.}$$

8. **DESIGN OF GIRDERS.**—The depth of the web plate will be taken as 54 in. (see paragraph 4). The thickness of the web plate is influenced by two factors:

1. The shearing stresses.
2. The web must be thick enough to insure a practicable rivet spacing in the flanges, at the end of girder.

Neither of these factors are likely to exert much influence on a girder as small as the one under consideration.

Using a web plate 54 in. × $\frac{5}{16}$ in. the actual unit shear on the gross section of the web is

$$56{,}300 \div 16.90 = 3{,}330 \text{ lb. per sq. in.}$$

The allowable unit stress is 10,000 lb. per sq. in.

If it be assumed for the present that the distance between the rivet lines of the top and bottom flanges is 3 in. less than the distance back to back of flange angles and that there will be but one row of rivets connecting the flange to the web plate, the rivet spacing in the flanges at the end of the girder from formula (8) will be

$$p = \frac{r \cdot h'}{V} = \frac{5{,}630 \times 51.5}{56{,}300} = 5.2 \text{ in.}$$

where $p$ = pitch of rivet in inches, $r$ = allowable stress on rivet, and $V$ = shear at section. From this calculation it is seen that a $\frac{5}{16}$ in. web plate is satisfactory as far as the flange rivet spacing is concerned. A web plate 54 in. × $\frac{5}{16}$ in. will be adopted.

The gross area of the compression flange should not be less than the gross area of the tension flange (§ 50). The allowable fiber stress in the bottom flange is 16,000 lb. per sq. in. on the net area (§ 37) and in the top flange is $16{,}000 - 150l/b$, where $l$ = the distance between lateral supports of the top flange $= 8 \times 12 = 96$ in. and $b$ = the width of the cover plate (§ 50). The value of $b$ can be more easily estimated after the tension flange is determined. Assuming the effective depth to be $1\frac{1}{2}$ in. less than the distance back to back of flange angles the net area required for the tension flange is found as follows (formula (1))

$$A_F' = \frac{M}{f \cdot h} = \frac{8{,}280{,}000}{16{,}000 \times 53} = 9.76 \text{ sq. in.}$$

Allowing one-eighth of the area of the web as flange area, the required net area of the tension flange is

$$9.76 - \tfrac{1}{8} \times 16.90 = 7.65 \text{ sq. in.}$$

Two angles $5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$ provide a net area of 7.96 sq. in. deducting one $\frac{7}{8}$ in. hole from each angle. This section will be used with the 5 in. legs outstanding.

The width of the cover plate must be at least $\frac{1}{12} \times 96 = 8$ in. The width of the tension flange is about 10 in.

A 10 in. cover plate will probably prove satisfactory for the compression flange. The allowable unit stress in this flange from § 50, is

$$f_c = 16{,}000 - 150\tfrac{96}{10} = 14{,}560 \text{ lb. per sq. in.}$$

The gross area required for the compression flange is (formula (1))

$$A_F = \frac{M}{f_c \cdot h} = \frac{8{,}280{,}000}{14{,}560 \times 53} = 10.72 \text{ sq. in.}$$

Allowing one-eighth of the area of the web, as flange area the required gross area of the compressive flange is

$$10.72 - 2.11 = 8.61 \text{ sq. in.}$$

The area provided by a 10 in. × $\frac{5}{16}$ in. cover plate and 2 angles 4″ × 4″ × $\frac{3}{8}$″ is 8.86 sq. in., and this section will be used. The edge distance on the cover plate is about 2½ in. Allowable 8 × $\frac{5}{16}$ = 2½ in.

The gross area of the tension flange is 8.94 sq. in., and of the compression flange 8.86 sq. in., excluding the portion of the web.

The effective depth of the section adopted is 53.4 in., which is slightly larger than the assumed depth of 53.0 in., so no revision will be made.* The distance between gage lines is 50 in.

The portion of the moment carried by the flanges is 8.86 + 10.97 = 0.81. The required spacing of the rivets between the web and flanges is

$$p = \frac{r \cdot h}{0.81V} = \frac{5,630 \times 50}{0.81 \times 56,300} = 6.17 \text{ in.}$$

The maximum spacing allowed is 6 in. but this is longer than should be used. A spacing of 5 in. will be used throughout the entire length of the top and bottom flanges. The required spacing of rivets between the cover plate and flange angles is much greater than for the case just considered. The maximum allowed is 16 × $\frac{5}{16}$ = 5 in. This spacing will be used.

The end stiffeners should have an area sufficient to carry the total end shear by column action. There will be two pairs of end stiffeners at each end of each girder. Sliding bearings are to be used so the pair of stiffeners towards the center of the girder should be designed for $\frac{3}{4}$ of the shear or 42,200 lb. Only the area of the outstanding legs should be considered as effective at the ends of the stiffeners because of the poor bearing of the other leg on the fillet of the flange angle. The area required is 42,200 ÷ 16,000 = 2.64 sq. in. The value 16,000 is used without reduction for there is no column action at the ends of stiffeners. At other points along the stiffeners the full section can be used, but the allowable stress must be reduced by the column formula. The case just figured will evidently control, however. Two angles 4″ × 3″ × $\frac{3}{8}$″ provide an area of 8 × $\frac{3}{8}$ = 3.00 sq. in. so will be used for the end stiffeners nearer the center of the girder. The other pair of end stiffeners will be made of two angles 4″ × 3″ × $\frac{5}{16}$″. A 10 in. × ¼ in. plate will be riveted to these to improve the appearance of the end of the girder.

The outstanding leg of the intermediate stiffeners must not be less than 1/30 × 54 + 2 = 3.8 in. All intermediate stiffeners will be made of angles 4″ × 3″ × $\frac{5}{16}$″ placed in pairs, with the 4 in. leg outstanding.

The stress carried by the end stiffeners nearest the center of the girder is 42,200 lb. as determined above. Enough rivets must be used between the angles and the web to transmit this stress to the web in double shear or bearing. The number required is 42,200 ÷ 5,630 = 8. The rivets will be spaced about 5 in. This will provide about 10 rivets for this case. The rivets in all stiffeners will have the same spacing.

The maximum length of $\frac{5}{16}$ in. plate 60 in. wide which can be obtained at the mills is 460 in. or 38 ft. 4 in., so the web plate will have to be spliced. This splice might be located at the center of the girder, but it is better practice to splice at points where there is an excess of flange area. Each girder will be spliced near the third points at a distance of about 17 feet from the end of the girder and at stiffeners.

The dead load per foot per girder is

$$120,750 \div 2 \times 50 = 1,210 \text{ lb.}$$

The live load per foot per girder is 1.287 × 90 × 18 + 2 = 1,040 lb. including impact.
The dead load shear at the point of splice is

$$V_D = 1,210 \times \tfrac{49}{2} - 1,210 \times 17 = 9,000 \text{ lb.}$$

The dead load moment at the point of splice is

$$M_D = \tfrac{1}{2} \times 1{,}210 \times 49 \times 17 - \tfrac{1}{2} \times 1{,}210 \times 17^2 = 329{,}000 \text{ ft.-lb.}$$

The maximum live load shear will occur with the uniform load covering the girder up to the point of splice, and is

$$V_L = \frac{1{,}040 \times 32 \times 16}{49} = 10{,}850 \text{ lb.}$$

The maximum live load moment occurs with the entire span loaded, and is

$$M_L = \tfrac{1}{2} \times 1{,}040 \times 49 \times 17 - \tfrac{1}{2} \times 1{,}040 \times 17^2 = 282{,}000 \text{ ft.-lb.}$$

The total shear at the section is

$$V = 9{,}000 + 10{,}850 = 19{,}850 \text{ lb.}$$

The total moment at the section is

$$M = 329{,}000 + 282{,}000 = 611{,}000 \text{ ft.-lb.}$$

The amount of the moment carried by the web is equal to the ratio of the web area counted as flange area, to the total area of one flange and, considering the tension flange is

$$\frac{2.11}{2.11 + 7.96} \times 611{,}000 = 121{,}000 \text{ ft.-lb.}$$

The type of splice shown in Fig. 12 will be used. The number of shop rivets required by shear alone is determined by bearing on the $\frac{5}{16}$ in. web and is $19{,}850 \div 5{,}630 = 4$. This number

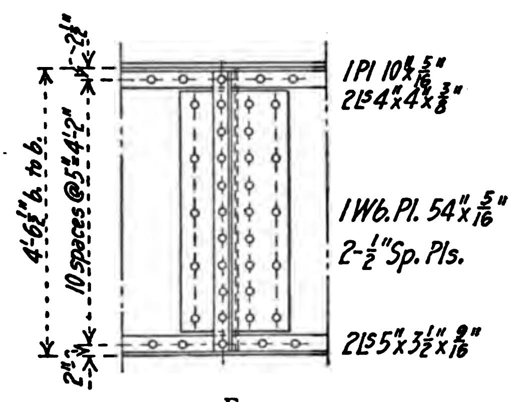



FIG. 12.

must be materially increased to take care of the moment. There should be two rows of rivets on each side of the splice. The arrangement shown in Fig. 12 represents the minimum number of rivets which can be conveniently used in this case

$$\Sigma d^2 = (5^2 + 2 \times 10^2 + 15^2 + 2 \times 20^2) = 1{,}250 \text{ in}^2.$$

$$r = \sqrt{\left(\frac{V}{2n}\right)^2 + \left(\frac{M \cdot d_n}{2 \cdot \Sigma d^2}\right)^2} = \sqrt{\left(\frac{19{,}850}{14}\right)^2 + \left(\frac{121{,}000 \times 20}{2{,}500}\right)^2}$$

$$= \sqrt{(1{,}420)^2 + (970)^2} = 1{,}720 \text{ lb.}$$

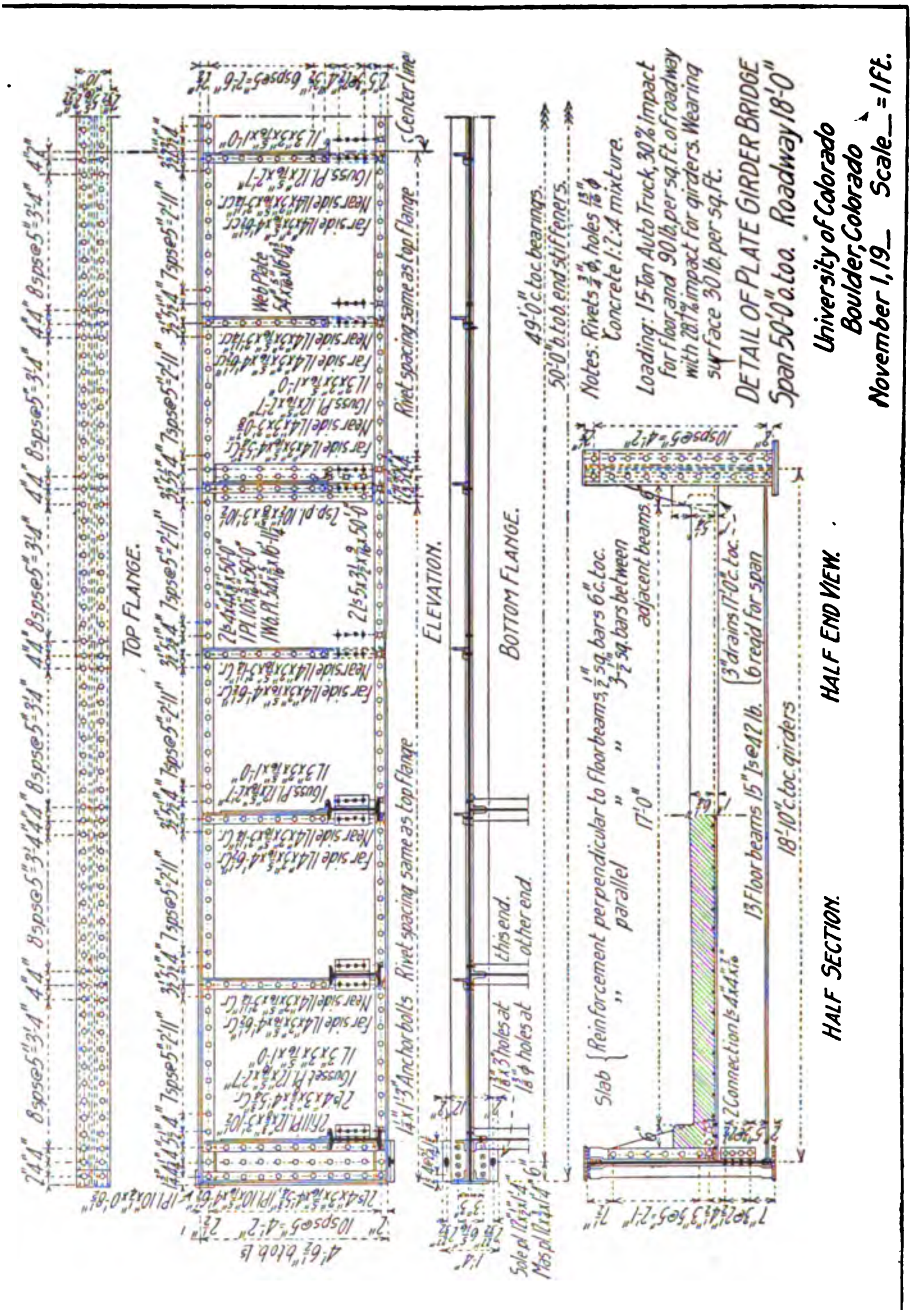

TOP FLANGE.
ELEVATION.
BOTTOM FLANGE.
Rivet spacing same as top flange
Center line
49'-0" c. to c. bearings.
50'-0" b. to b. end stiffeners.
Notes: Rivets 3/4" φ, holes 13/16" φ
Concrete 1:2:4 mixture.
Loading: 15-Ton Auto Truck, 30% Impact for floor and 90 lb. per sq. ft. of roadway with 28.1% impact for girders. Wearing surface 30 lb. per sq. ft.
DETAIL OF PLATE GIRDER BRIDGE
Span 50'-0" c. to c. Roadway 18'-0"
University of Colorado
Boulder, Colorado
November 1, 19__ Scale __ = 1 ft.
HALF END VIEW.
HALF SECTION.
13 Floor beams 15" I's @ 42 lb.
18'-10" c. to c. girders

FIG. 13.

Allowable bearing (Table 33, Appendix III) = 5,630 lb. This splice will be used as there is no place where it is advisable to reduce the number of rivets. The flange angles and cover plates can be obtained long enough for the entire span, so they will not have to be spliced.

9. **DESIGN OF END BEARINGS.**—Roller bearings are not required for spans less than 70 ft., so sliding bearings will be used for this girder. Slotted holes will be used in the sole plates to allow for a movement of at least $\frac{5}{8}$ in. at one end. The area of the wall plate must be at least

$$A = \frac{V}{f} = \frac{56,300}{600} = 94 \text{ sq. in.}$$

where $V$ = the maximum end reaction, and $f$ = allowable bearing on concrete abutments. The sizes of the sole plates and masonry plates will probably be determined by the detail adopted. The thickness of the sole plate will be taken as $\frac{5}{8}$ in. and the masonry plate as $\frac{3}{4}$ in.

The anchor bolts will be hacked bolts, $1\frac{1}{4}$ in. in diameter and 1 ft. 3 in. long.

10. **Detail Drawings.**—The detail drawings for this bridge are shown in Fig. 13.

# CHAPTER XIII.

## Design of Low Truss Highway Bridges.

**Introduction.**—Low truss highway bridges are used for spans of from 30 to 80 feet, and for special designs to 100 feet. The trusses may have either pin-connected or riveted joints. The trusses may have either half-hip, as in Fig. 1 or full slopes, as in Fig. 2, and may be either of the Warren type, as in Fig. 1, or of the Pratt type, as in Fig. 2. The cost is practically the same for the two types. Low truss highway bridges should always be made with riveted connections.

**Design of Riveted Trusses.**—The author's specifications for the Design of Steel Highway Bridges, Appendix I, contain the following requirements for the design of low truss bridges.

§ 3. *Types of Truss.*—Spans from 30 ft. to 80 ft.—Riveted plate girders or riveted low trusses for classes A, B, $E_1$, $E_2$ and $E_3$, and riveted low trusses for classes C, $D_1$ and $D_2$.

§ 54. *Low Trusses.*—Riveted low trusses shall have top chords composed of double web members with cover plate. The top chord shall be stayed against lateral bending by means of brackets or knee braces rigidly connected to the floorbeams at intervals not greater than twelve times the width of the cover plate. The posts shall be solid web members. The floorbeams shall be riveted, preferably above the lower chord. Pin-connected low truss bridges shall not be used.

General Specification for Steel Highway Bridges adopted 1918 by the Engineering Institute of Canada contains the following specifications with reference to the design of low truss highway bridges.

"Pony-truss bridges shall be of riveted type. Spans of 50 feet, center to center of bearings, or less, may have single-webbed trusses with T-chords; but all spans over 50 feet in length shall have double-webbed chords, and latticed or otherwise effectually stiffened web-members, unless otherwise specified by the engineer.

"In all pony-truss bridges, the floorbeams shall be rigidly connected to vertical truss-members; and stiffening gussets, as large as practicable without interfering with the roadway clearances, shall be provided. The vertical truss-members and the floorbeam connections thereto shall, when practicable, be proportioned to resist, at the specified unit-stresses, a lateral force applied at the top-chord of the truss, equal to 2 per cent of the maximum top-chord stress. When im-

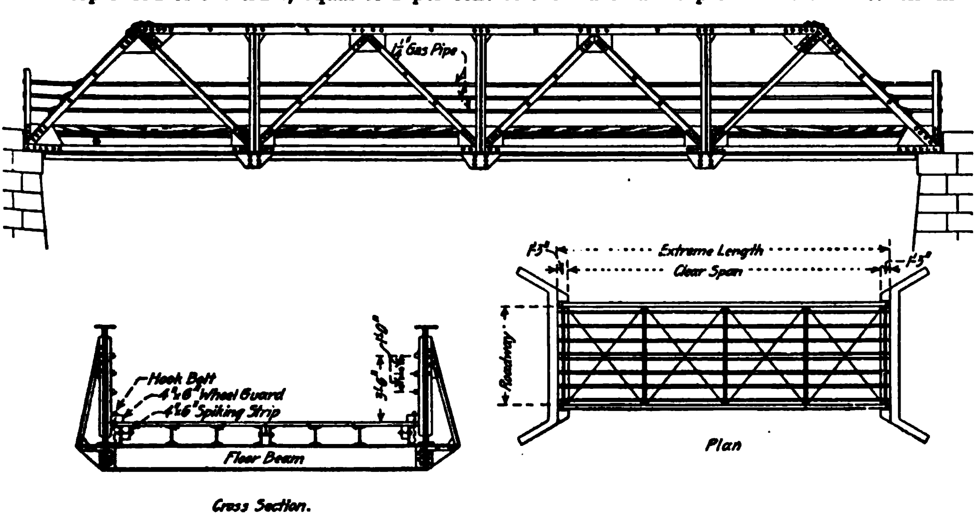



Fig. 1. Low Warren Riveted Highway Bridge. Gillette-Herzog Mfg. Co.

practicable to design the vertical truss-members sufficiently strong to meet this requirement, outside wing-braces shall be added."

The specifications of the Massachusetts Public Service Commission contain the following requirements for the design of riveted highway bridge trusses.

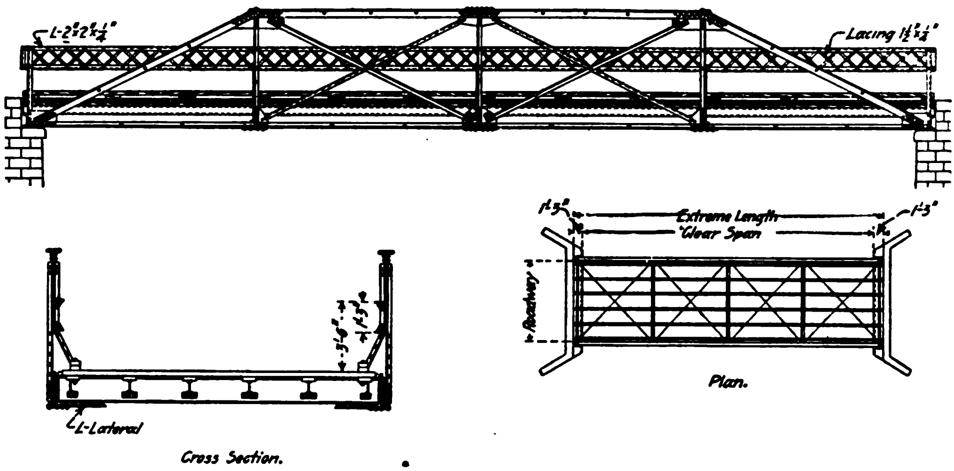



FIG. 2. LOW PRATT RIVETED HIGHWAY BRIDGE. AMERICAN BRIDGE COMPANY.

§ 70. *General Principles.*—Riveted trusses shall be so laid out that the centre of gravity lines (not gage lines) of each member meeting at a joint shall intersect at a point, as nearly as practicable. Only very small deviations will be permitted.

§ 71. Rivets taking stress out of any member, whether at a splice or at a joint, shall be arranged as nearly as practicable so that the center of gravity of the rivets shall be in the line

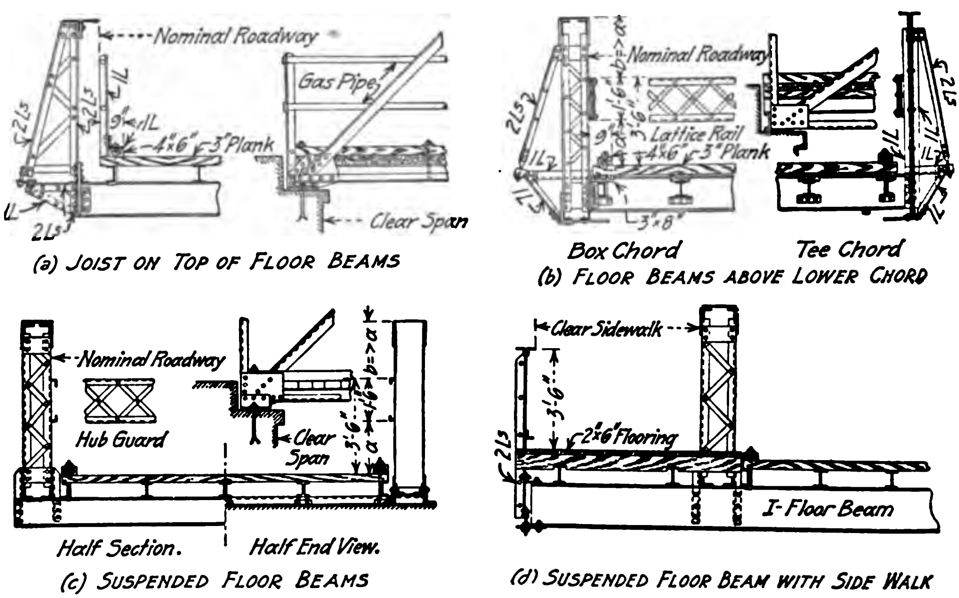



FIG. 3. DETAILS OF LOW TRUSS HIGHWAY BRIDGES. AMERICAN BRIDGE COMPANY.

of the center of gravity of the member, and symmetrically arranged, if practicable. Angles must generally be connected by both legs, using lug angles, with number of rivets in each leg proportional to area of leg.

§ 72. When stress is taken out of a member by rivets, the rivets must be arranged, by staggering or otherwise, so as to have as few rivets as practicable in a line parallel to the member; and the pitch of rivets must generally be as small as practicable.

§ 73. When stress is taken out of a single member into a gusset plate, and from this into another member, the rivets in the second member must not only have their center of gravity in the center of gravity line of this member (see § 71), but also as nearly as practicable in the center of gravity line of the first member.

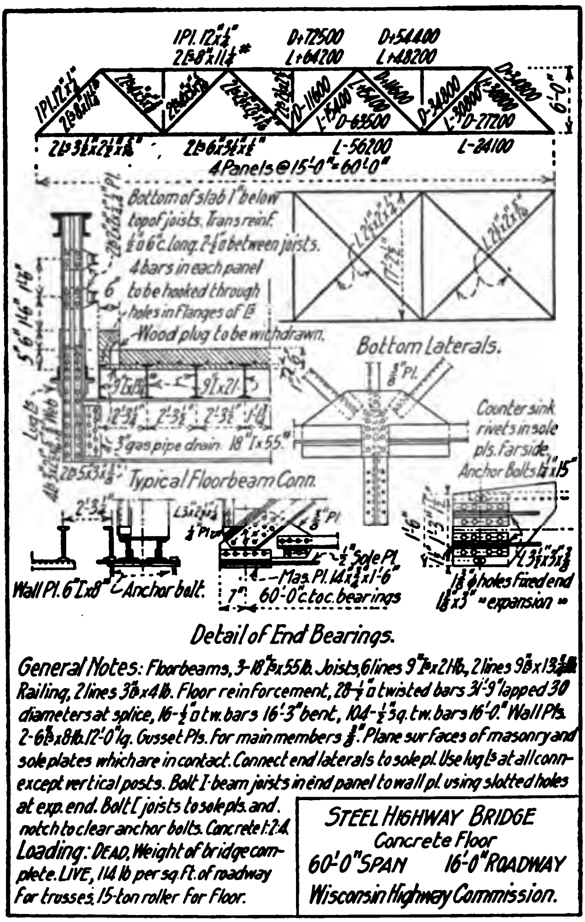



FIG. 4. LOW TRUSS STEEL HIGHWAY BRIDGE. WISCONSIN HIGHWAY COMMISSION.

§ 74. Any bending on riveted connections, due to necessary nonfulfillment of these general principles, is to be allowed for in proportioning the parts.

§ 75. *Sections.* For upper chord and end-post sections a T-shape will not be allowed. If two vertical plates are used, the bottom edges must be properly stiffened, as circumstances may dictate, either by tie plates, lacing with bent bars, or generally by the addition of angles with lacing of the usual kind.

§ 76. *Web Members.*—Web members must be double, and connected symmetrically to the chords.

§ 77. *Splices.*—Where a chord is spliced at a joint, and the gusset plate used as a cover, only that part of the gusset in contact with the chord shall be allowed for.

**EXAMPLES.**—A four-panel Warren truss highway bridge, designed by the Gillette-Herzog Mfg. Co., is shown in Fig. 1. The chords and the diagonal web members are made of two angles placed back to back, forming a T-section, while the posts are made of two angles "starred." The floorbeams are riveted below the lower chord and are rolled I-beams. The joists are carried directly on the tops of the floorbeams and are composed of four channels and four I-beams placed as shown. The details of the floor are clearly shown. The lower laterals are made of single angles with riveted connections. The upper chord is braced at the panel points with angle braces. This type of bridge should be used only for very light highway bridges.

TABLE I.

DATA ON STANDARD STEEL LOW TRUSS HIGHWAY BRIDGES.
WISCONSIN HIGHWAY COMMISSION.

| Span, Ft. | Height of Truss, Feet. Roadway. | | Number of Panels. | Weight of Structural Steel in Trusses, Floorbeams and Lateral Bracing. | |
|---|---|---|---|---|---|
| | 16 Ft. | 18 Ft. | | 16 Ft. Roadway, Lb. | 18 Ft. Roadway, Lb. |
| 35 | 4.5 | 4.5 | 3 | 7,210 | 7,680 |
| 40 | 5.0 | 5.0 | 3 | 7,980 | 8,660 |
| 45 | 5.25 | 5.25 | 3 | 9,980 | 10,665 |
| 50 | 5.5 | 5.5 | 4 | 11,970 | 13,180 |
| 55 | 6.0 | 6.0 | 4 | 14,570 | 16,460 |
| 60 | 6.0 | 6.5 | 4 | 16,900 | 18,250 |
| 65 | 6.5 | 7.0 | 5 | 20,450 | 22,540 |
| 70 | 7.0 | 7.5 | 5 | 22,730 | 24,470 |
| 75 | 7.5 | 8.0 | 5 | 24,530 | 28,100 |
| 80 | 8.0 | 8.5 | 5 | 27,880 | 30,840 |
| 85 | 8.5 | 9 | 6 | 32,920 | 35,360 |
| Railing per lineal foot | | | | 17 lb. | 17 lb. |
| Reinforcing per lineal foot | | | | 40 lb. | 44 lb. |
| Concrete per lineal foot | | | | 0.30 cu. yd. | 0.34 cu. yd. |
| Concrete slabs 6 in. thick. Reinforcing transverse ½ in. sq. twisted bars spaced 6 in. centers, longitudinal ½ in. sq. twisted bars spaced two between joists. | | | | | |

A four-panel Pratt low truss highway bridge, designed by the American Bridge Company, is shown in Fig. 2. The upper and the lower chords and the posts are made of two angles placed back to back, forming a T-section. The diagonals are single angles acting in tension, only. The trusses are braced by bending one of the angles of the posts. The floorbeams are riveted to the posts above the lower chords, and are rolled I-beams. The joists are carried on shelf angles riveted to the webs of the floorbeams as shown. The lower lateral systems are made of single angles with riveted connections.

Details of a riveted low truss highway bridge with the floorbeams riveted below the lower chords, as designed by the American Bridge Company, are shown in (*a*), Fig. 3. The end shoe is bolted to the bridge seat by means of anchor bolts. The holes in the bearing plates of the shoes should be slotted at one end to permit movement due to changes in temperature. Sliding plates should be provided at the expansion end; the surfaces of the bearing and sliding plates in contact being planed.

Details of riveted low truss highway bridges with box- and with tee-chords, and with floorbeams riveted below the lower chords, as designed by the American Bridge Company, are shown in (*b*), Fig. 3. Details of a riveted low truss highway bridge with box-chords and with suspended floorbeams are shown in (*c*), Fig. 3. It will be noted that no side braces are provided in this

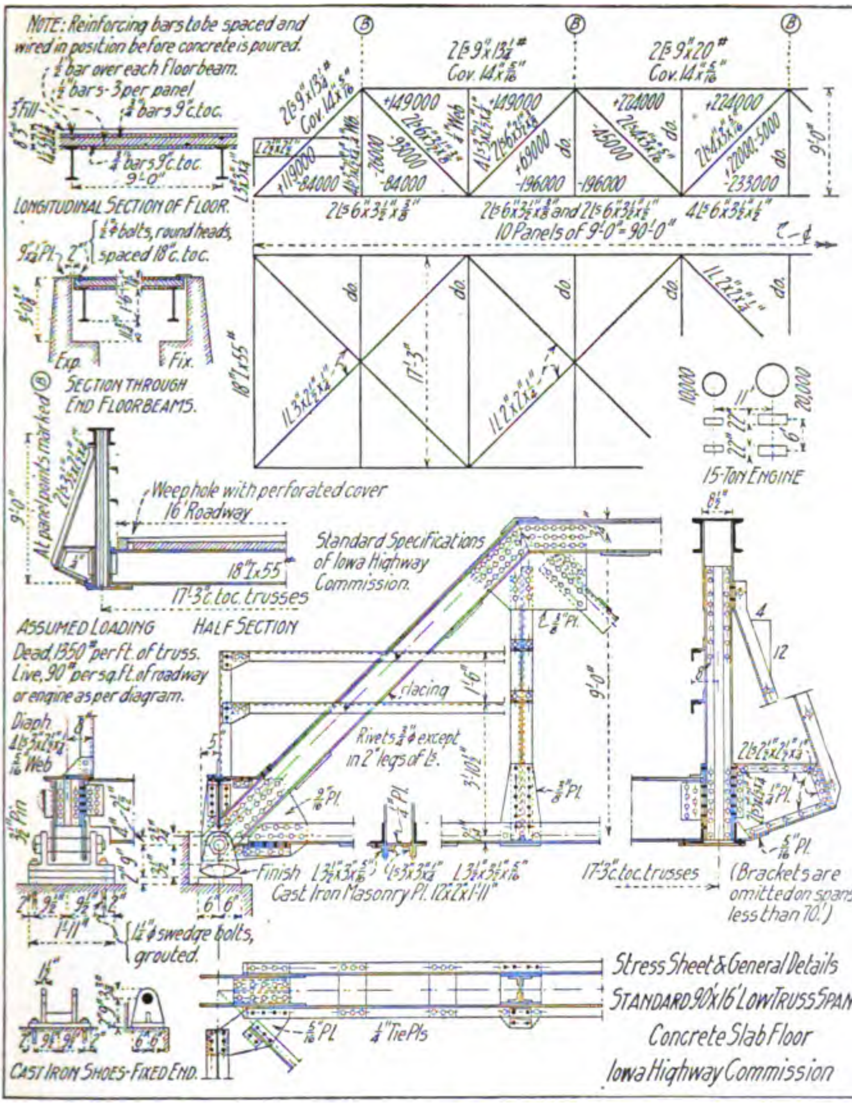


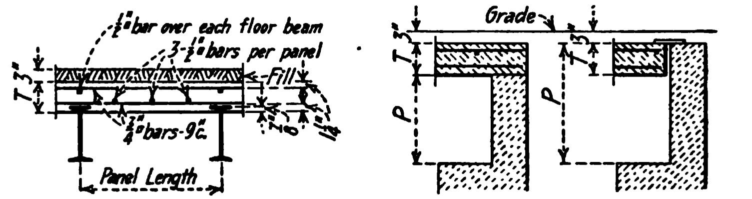

FIG. 5. LOW TRUSS STEEL HIGHWAY BRIDGE, WITH SLAB FLOOR. IOWA HIGHWAY COMMIS

design. The same method of suspending the floorbeams is shown in (*d*), Fig. 3, for a low bridge with sidewalks. Where side braces are not used the posts should be made wider where braces are used.

Details of a 60-ft. span standard low truss highway bridge designed by the Wisconsin way Commission are given in Fig. 4. The upper chords are composed of two channels 8 in. @ lb., and a 12 in. by ¼ in. cover plate. The joists are carried on the tops of the floorbeams, the floorbeams are riveted to the posts below the lower chords. This bridge was designe a 15-ton road roller and for a live load of 114 lb. per sq. ft., both without impact, in addition t

## TABLE II.

### STANDARD STEEL LOW TRUSS SPANS—CONCRETE FLOOR WITHOUT STRINGERS. GENERAL DATA AND ESTIMATED QUANTITIES. IOWA HIGHWAY COMMISSION.




SECTIONS OF BRIDGE SEATS
Spans 35' to 65' Incl. Spans 70' to 100' Incl.

| Span and Roadway. | Height of Truss. | | Number of Panels. | Center to Center of Trusses. | | Thickness of Floor, T. | Height of Parapet, P. | | Structural Steel. | Floor. | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | Reinforcing. | Concrete |
| ft. | ft. | in. | | ft. | in. | in. | ft. | in. | lb. | lb. | cu. yd. |
| 35 × 16 | 6 | 0 | 4 | 17 | 0 | 8 | 1 | $7\frac{15}{16}$ | 12,200 | 3,470 | 15.2 |
| 35 × 18 | 6 | 0 | 4 | 19 | 0 | 8 | 1 | $7\frac{15}{16}$ | 12,800 | 3,880 | 17.0 |
| 40 × 16 | 6 | 0 | 5 | 17 | 0 | $7\frac{1}{2}$ | 1 | 8 | 15,300 | 3,940 | 16.1 |
| 40 × 18 | 6 | 0 | 5 | 19 | 0 | $7\frac{1}{2}$ | 1 | 8 | 16,100 | 4,400 | 18.1 |
| 45 × 16 | 6 | 6 | 5 | 17 | 0 | 8 | 1 | 8 | 16,700 | 4,340 | 19.2 |
| 45 × 18 | 6 | 6 | 5 | 19 | 0 | 8 | 1 | $8\frac{1}{16}$ | 17,500 | 4,860 | 21.5 |
| 50 × 16 | 6 | 6 | 6 | 17 | 3 | $7\frac{1}{2}$ | 1 | $8\frac{1}{8}$ | 19,900 | 4,820 | 19.9 |
| 50 × 18 | 6 | 6 | 6 | 19 | 3 | $7\frac{1}{2}$ | 1 | $8\frac{3}{16}$ | 21,000 | 5,370 | 22.3 |
| 55 × 16 | 6 | 6 | 6 | 17 | 3 | 8 | 1 | $8\frac{1}{4}$ | 21,400 | 5,230 | 23.2 |
| 55 × 18 | 6 | 6 | 6 | 19 | 3 | 8 | 1 | $8\frac{3}{16}$ | 23,100 | 5,830 | 26.0 |
| 60 × 16 | 7 | 0 | 7 | 17 | 3 | 8 | 1 | $8\frac{1}{8}$ | 25,500 | 5,850 | 25.2 |
| 60 × 18 | 7 | 0 | 7 | 19 | 3 | 8 | 1 | $8\frac{3}{16}$ | 27,200 | 6,540 | 28.3 |
| 65 × 16 | 7 | 0 | 8 | 17 | 3 | $7\frac{1}{2}$ | 1 | $8\frac{3}{16}$ | 29,100 | 6,310 | 25.5 |
| 65 × 18 | 7 | 0 | 8 | 19 | 3 | $7\frac{1}{2}$ | 1 | $8\frac{1}{4}$ | 31,000 | 7,060 | 28.7 |
| 70 × 16 | 7 | 0 | 8 | 17 | 3 | 8 | 3 | $0\frac{1}{2}$ | 34,700 | 6,580 | 28.5 |
| 70 × 18 | 7 | 0 | 8 | 19 | 3 | 8 | 3 | $0\frac{9}{16}$ | 36,900 | 7,350 | 32.0 |
| 75 × 16 | 7 | 6 | 9 | 17 | 3 | $7\frac{1}{2}$ | 3 | $0\frac{1}{8}$ | 39,400 | 7,040 | 28.6 |
| 75 × 18 | 7 | 6 | 9 | 19 | 3 | $7\frac{1}{2}$ | 2 | $11\frac{13}{16}$ | 41,700 | 7,870 | 32.1 |
| 80 × 16 | 8 | 0 | 10 | 17 | 3 | $7\frac{1}{2}$ | 3 | $0\frac{1}{8}$ | 43,800 | 7,500 | 30.5 |
| 80 × 18 | 8 | 0 | 10 | 19 | 3 | $7\frac{1}{2}$ | 2 | $11\frac{7}{8}$ | 46,600 | 8,380 | 34.3 |
| 85 × 16 | 8 | 6 | 10 | 17 | 3 | 8 | 3 | $0\frac{5}{16}$ | 47,100 | 7,920 | 34.5 |
| 85 × 18 | 8 | 6 | 10 | 19 | 3 | 8 | 3 | $0\frac{7}{16}$ | 49,400 | 8,840 | 38.8 |
| 90 × 16 | 9 | 0 | 10 | 17 | 3 | 8 | 3 | $0\frac{3}{8}$ | 50,200 | 8,490 | 36.5 |
| 90 × 18 | 9 | 0 | 10 | 19 | 3 | 8 | 3 | $0\frac{15}{16}$ | 52,800 | 9,490 | 41.0 |
| 95 × 16 | 9 | 6 | 10 | 17 | 3 | 8 | 3 | $0\frac{15}{16}$ | 52,400 | 8,910 | 38.5 |
| 95 × 18 | 9 | 6 | 10 | 19 | 3 | 8 | 3 | 3 | 59,200 | 9,960 | 43.2 |
| 100 × 16 | 10 | 0 | 10 | 17 | 3 | $8\frac{1}{2}$ | 3 | $1\frac{7}{16}$ | 57,200 | 9,310 | 43.0 |
| 100 × 18 | 10 | 0 | 10 | 19 | 4 | $8\frac{1}{2}$ | 3 | $3\frac{1}{2}$ | 64,000 | 10,400 | 48.3 |

dead load. These bridges could be very much improved (1) by riveting the floorbeams above the lower chords; (2) by the use of solid webbed posts, and (3) by the use of side braces or brackets. Data on Wisconsin Highway Commission standard low truss highway bridges are given in Table I.

**Details of a 90-ft. span standard low truss highway bridge without joists, as designed by the Iowa Highway Commission are given in Fig. 5. These bridges have a concrete slab floor carried directly on the floorbeams. The details of this bridge are well worked out and give an excellent structure. Standard spans from 35 ft. to 65 ft. rest on sliding plates, while spans from 70 ft. to 100 ft. have one end on a rocker. In spans from 35 ft. to 65 ft. the side braces are omitted. Data on the low truss highway bridges, the details of which are shown in Fig. 5, are given in Table II.**

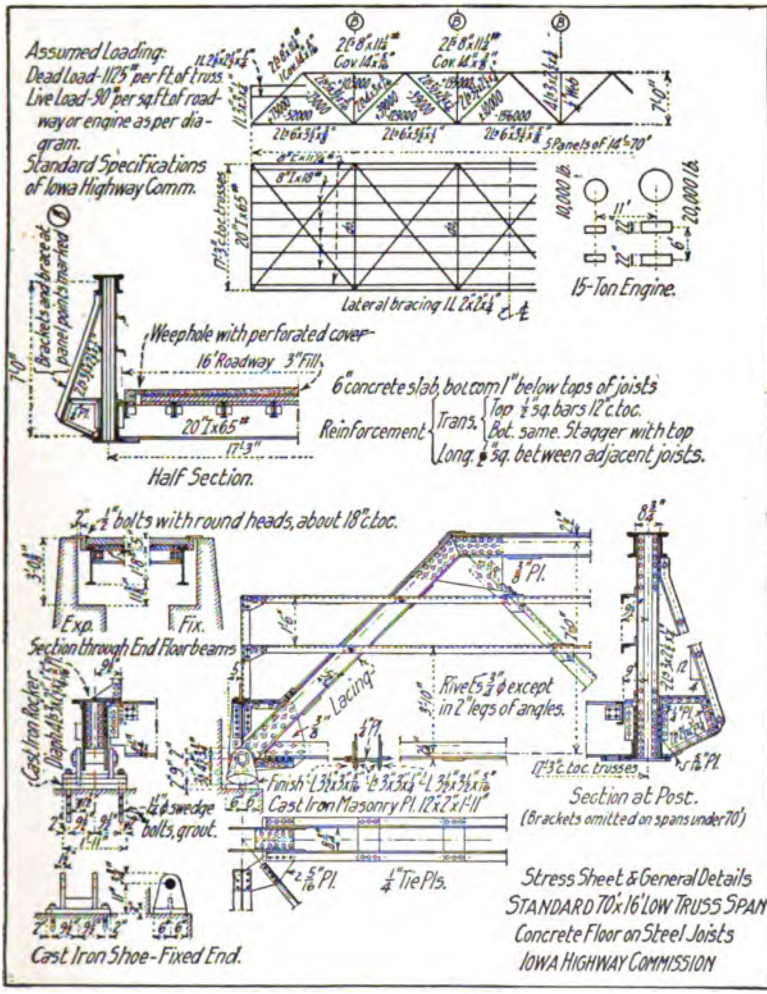


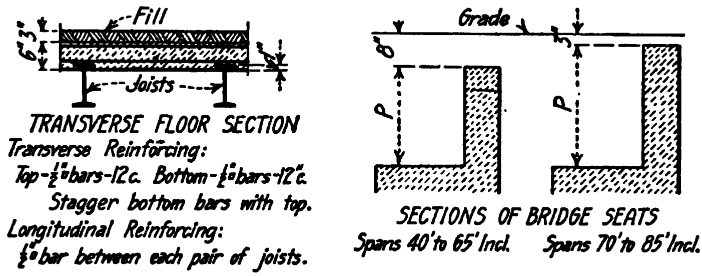

FIG. 6. LOW TRUSS STEEL HIGHWAY BRIDGE, CONCRETE FLOOR ON STEEL STRINGERS. IOWA HIGHWAY COMMISSION.

**Details of a 70-ft. span low truss highway bridge with joists, as designed by the Iowa Highway Commission are given in Fig. 6. These bridges have a concrete slab floor carried on steel joists. The steel I-beam joists are framed into the floorbeams by coping the joists so that the top flanges of both floorbeams and joists are on the same level. Standard spans of 35 ft. to 65 ft. rest on**

sliding plates, while spans from 70 ft. to 85 ft. rest on rockers. In spans from 35 ft. to 65 ft. the side braces are omitted. Data on the low truss highway bridge shown in Fig. 6, are given in Table III.

## TABLE III.

STANDARD STEEL LOW TRUSS SPANS—WITH STEEL STRINGERS AND CONCRETE FLOOR.
GENERAL DATA AND ESTIMATED QUANTITIES.
IOWA HIGHWAY COMMISSION.




| Span and Roadway. | Height of Truss. | Number of Panels | Center to Center of Trusses. | Height of Parapet, P. | Structural Steel. | | | Floor. | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | Trusses, Bracing, Floorbeams, etc. | Joists. | Total. | Reinforcing. | Concrete. |
| ft. | ft. in. | | ft. in. | ft. in. | lb. | lb. | lb. | lb. | cu. yd. |
| 40 × 16 | 6 0 | 3 | 17 3 | 1 8 13/16 | 12,500 | 5,700 | 18,200 | 1,390 | 13.0 |
| 40 × 18 | 6 0 | 3 | 19 3 | 1 10 7/8 | 13,800 | 6,500 | 20,300 | 1,580 | 14.6 |
| 45 × 16 | 6 6 | 3 | 17 3 | 1 10 15/16 | 14,600 | 6,300 | 20,900 | 1,560 | 14.5 |
| 45 × 18 | 6 6 | 3 | 19 3 | 1 10 13/16 | 15,300 | 7,200 | 22,500 | 1,760 | 16.3 |
| 50 × 16 | 6 6 | 3 | 17 3 | 1 11 | 16,000 | 8,100 | 24,100 | 1,710 | 15.9 |
| 50 × 18 | 6 6 | 3 | 19 3 | 1 11 | 17,500 | 9,200 | 26,700 | 1,930 | 17.9 |
| 55 × 16 | 6 6 | 4 | 17 3 | 1 8 13/16 | 18,600 | 7,800 | 26,400 | 1,870 | 17.4 |
| 55 × 18 | 6 6 | 4 | 19 3 | 1 11 | 20,600 | 8,900 | 29,500 | 2,120 | 19.6 |
| 60 × 16 | 7 0 | 4 | 17 3 | 1 11 | 21,300 | 8,500 | 29,800 | 2,040 | 18.9 |
| 60 × 18 | 7 0 | 4 | 19 3 | 1 11 | 22,400 | 9,600 | 32,000 | 2,310 | 21.2 |
| 65 × 16 | 7 0 | 4 | 17 3 | 1 11 1/16 | 23,000 | 10,600 | 33,600 | 2,200 | 20.4 |
| 65 × 18 | 7 0 | 4 | 19 3 | 1 11 1/8 | 25,700 | 12,000 | 37,700 | 2,490 | 22.9 |
| 70 × 16 | 7 0 | 5 | 17 3 | 3 0 3/8 | 31,700 | 9,900 | 41,600 | 2,300 | 21.2 |
| 70 × 18 | 7 0 | 5 | 19 3 | 3 0 5/16 | 32,900 | 11,300 | 44,200 | 2,600 | 23.8 |
| 75 × 16 | 7 6 | 5 | 17 3 | 3 0 5/16 | 33,000 | 10,600 | 43,600 | 2,460 | 22.7 |
| 75 × 18 | 7 6 | 5 | 19 3 | 3 0 5/16 | 35,200 | 12,000 | 47,200 | 2,790 | 25.5 |
| 80 × 16 | 8 0 | 5 | 17 3 | 3 0 5/16 | 35,200 | 13,000 | 48,200 | 2,620 | 24.2 |
| 80 × 18 | 8 0 | 5 | 19 3 | 3 0 7/16 | 38,700 | 14,700 | 53,400 | 2,970 | 27.2 |
| 85 × 16 | 8 6 | 5 | 17 3 | 3 0 5/16 | 37,600 | 13,700 | 51,300 | 2,780 | 25.7 |
| 85 × 18 | 8 6 | 5 | 19 3 | 3 0 7/16 | 41,100 | 15,600 | 56,700 | 3,150 | 28.9 |

Details of a low riveted truss bridge with reinforced concrete floor as designed by the Michigan Highway Commission are given in Fig. 7. The upper chord is braced by gusset plates on the inside of each post. The expansion end of the bridge rests on 6-in. segmental steel rollers. The Commission has prepared standard plans for spans of from 50 ft. to 100 ft. by 5 ft. intervals.

The riveted low truss highway bridge with an inclined upper chord shown in Fig. 8, is built by the American Bridge Company for locations requiring an artistic, serviceable bridge at a moderate cost. This bridge has been built with six panels and with spans of 90, 96 and 102 feet.

The bridge in Fig. 8 has a 20-ft. roadway and was designed for a dead load of 930 lb. per lineal foot of bridge, and a live load of 2,400 lb. per lineal foot of bridge. The total weight of the steel in this bridge, exclusive of joists and fence is, approximately, 57,000 lb. The floorbeams are rolled I-beams and are riveted below the chords. The top chords are made of two channels

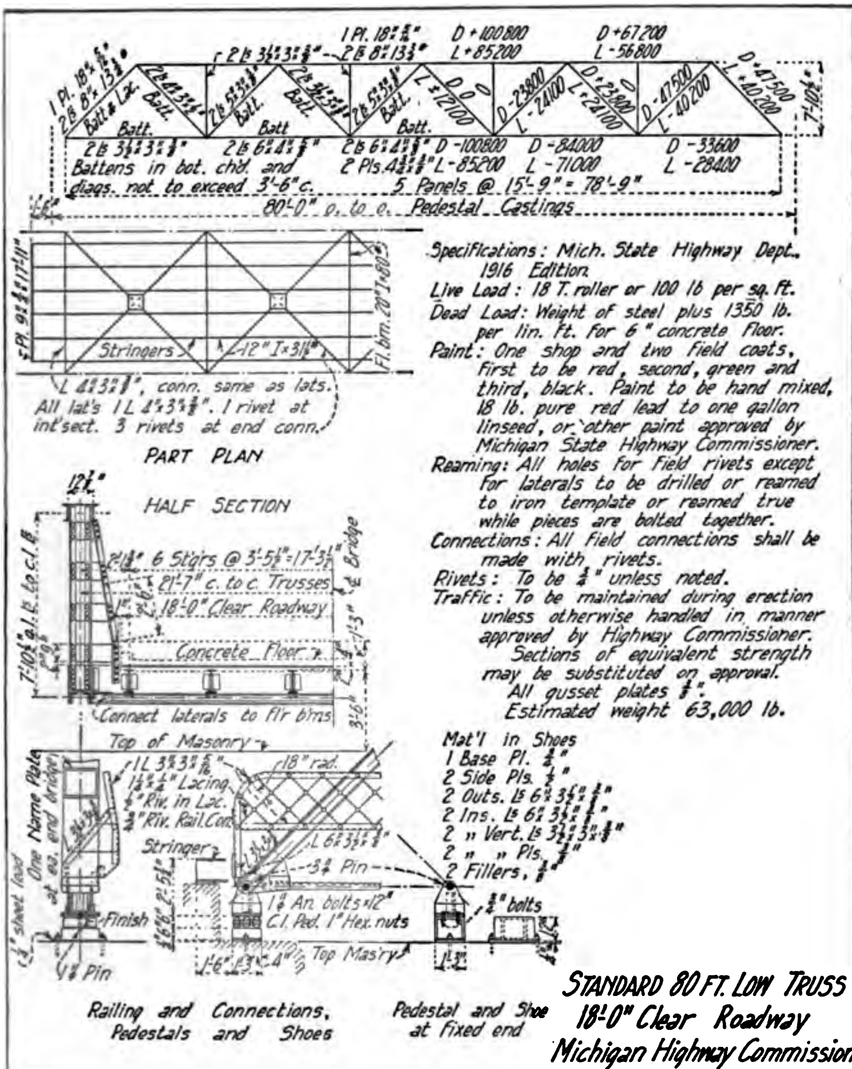



FIG. 7. LOW TRUSS STEEL HIGHWAY BRIDGE. MICHIGAN HIGHWAY COMMISSION.

with a top cover plate, the lower edges of the channels being fastened together with tie plates—lacing is much better practice. The bottom chord is composed of two angles, with tie plates—tie plates are all right for this member. The web members are made of 2 or 4 angles laced as shown. Rods, not shown, are used for the lower lateral system.

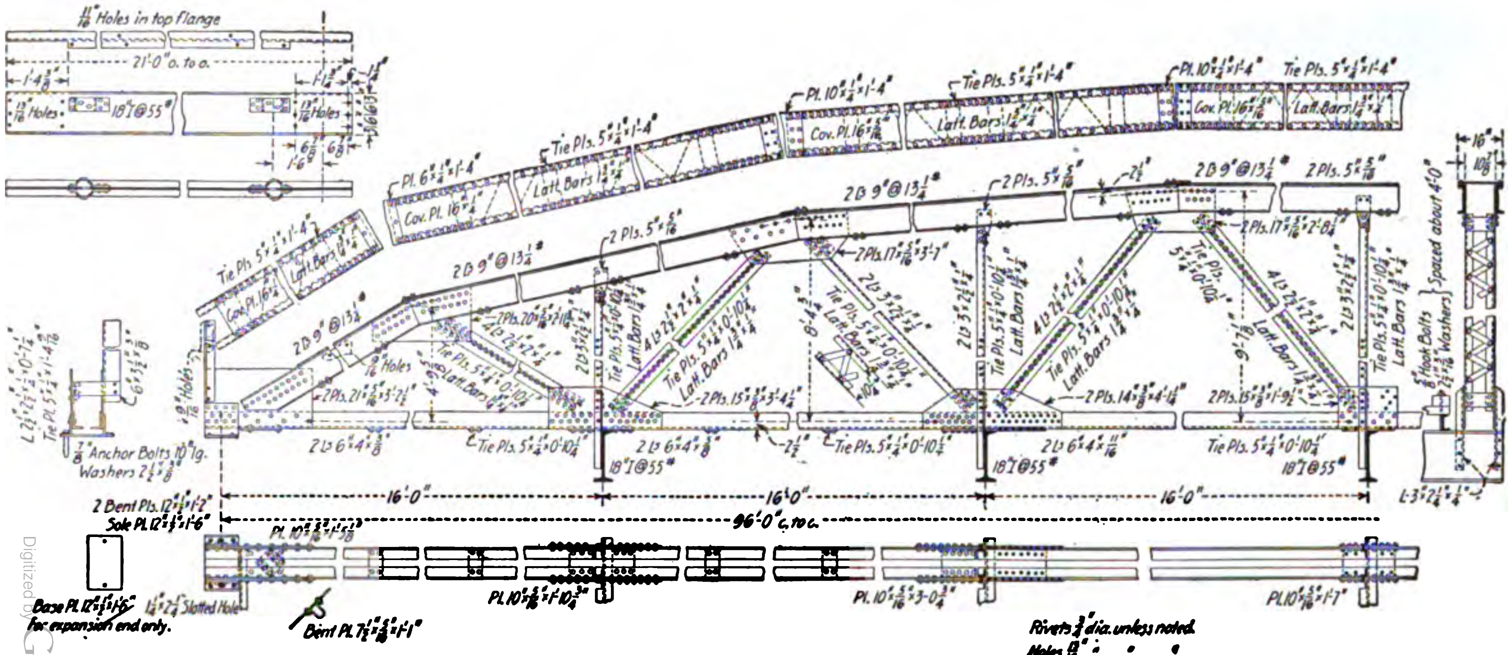



FIG. 8. DETAILS OF LOW TRUSS RIVETED HIGHWAY BRIDGE WITH INCLINED CHORDS. AMERICAN BRIDGE COMPANY.

**PIN-CONNECTED LOW TRUSS BRIDGES.**—Pin-connected low truss highway bridges are commonly built of the Pratt type, with either half-hip or full slope end-posts. The upper chords of pin-connected low truss highway bridges are made of two channels and a top cover plate, or of two channels laced; the posts are usually made of four angles laced or battened; while the tension members are made of rods or eye-bars. The posts and the chords should be made very wide and should be securely fastened to the floorbeams, or side braces should be used. The details of the American Bridge Company's half-hip, low truss Pratt highway bridge with the floorbeams riveted below the lower chords are shown in (*a*), Fig. 9; while details of a full slope Pratt highway bridge with floorbeams riveted below the lower chords are shown in (*b*), Fig. 9. Rolled beams are used for floorbeams, while the lower laterals are made of rods with screw ends.

The principal objection to pin-connected low truss bridges is that the vertical trusses are usually not sufficiently braced, and lack lateral stability. The "fish-bellied" truss bridge shown in Fig. 10 with the floorbeams riveted above the lower chords, is a decided improvement upon the usual type of low truss bridge. This bridge is very rigid and makes a very satisfactory structure. In Fig. 10 the lower chord pins, beginning with the left end, are called $L_0$, $L_1$, $L_2$, etc., while the upper chord pins are called $U_1$, $U_2$, etc., as shown. The top chords and end-posts are made of two channels and a top cover plate, with tie plates on the bottom of the member. The posts are made of four angles; eye-bars are used for the lower chords and main diagonals, while rods are used for the main ties and the diagonals in the middle panels. The joists are carried directly on the tops of the floorbeams.

**TEMPERATURE CHANGES.**—The expansion ends of low truss bridges should be placed on sliding plates or be carried on rollers or rockers. Rollers are not commonly used for low truss highway bridges having a span less than 70 ft. Details of rockers and rollers are given in Chapter XV. Details of rockers are given in Figs. 5, 6 and 15.

**WEIGHT OF LOW TRUSS BRIDGES.**—The weights of low truss highway bridges as designed by the Wisconsin Highway Commission are given in Table I, and as designed by the Iowa Highway Commission are given in Tables II and III. Formulas for weights of low truss highway bridges are given in Chapter IX.

**LENGTH OF SPAN.**—The American Bridge Company's standards include the following lengths of span for the different types of low truss bridges:

TABLE IV.

LOW TRUSS SPANS USED BY AMERICAN BRIDGE COMPANY.

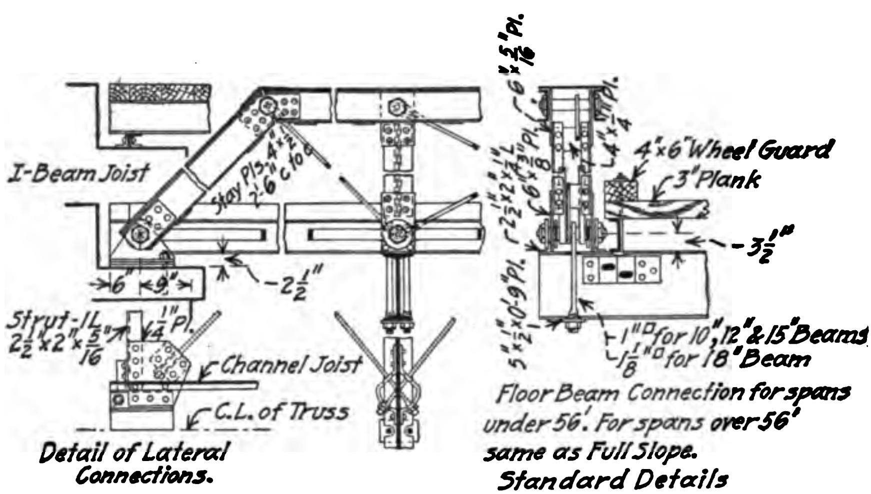
| Type of Truss. | Span in Feet. |
|---|---|
| Low Warren riveted truss with parallel chords, plate and two angles | 36 to 85 |
| Low Warren riveted trusses with inclined chords | 90 to 102 |

TABLE V.

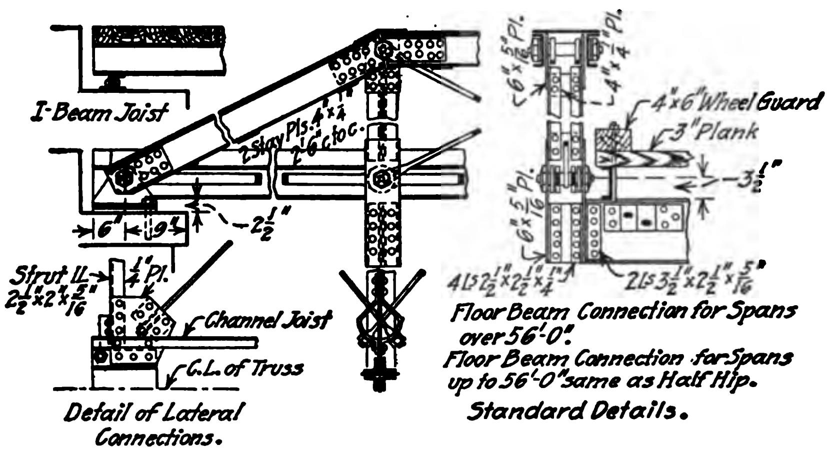
DEPTHS OF LOW RIVETED TRUSSES.

AMERICAN BRIDGE COMPANY.

| Span, Feet. | Number of Panels. | Ratio of Depth to Panel Length. |
|---|---|---|
| 36 to 45 | 3 | 0.35 |
| 48 to 60 | 4 | 0.40 |
| 65 to 85 | 5 | 0.50 |
| 90 to 102 | 6 | 0.30, 0.525, and 0.60 |




(a) DETAILS OF LOW PRATT, PIN CONNECTED TRUSS, HALF HIP.




(b) DETAILS OF LOW PRATT, PIN CONNECTED TRUSS, FULL SLOPE.

FIG. 9. LOW PRATT PIN-CONNECTED HIGHWAY BRIDGES.
AMERICAN BRIDGE COMPANY.

The Gillette-Herzog Mfg. Company's standards for riveted Warren low truss bridges included spans from 32 to 75 feet. The economical limit for low truss spans is at 75 or 80 feet.

**DEPTH OF TRUSS.**—The American Bridge Company's standards for low truss bridges are given in Table V.

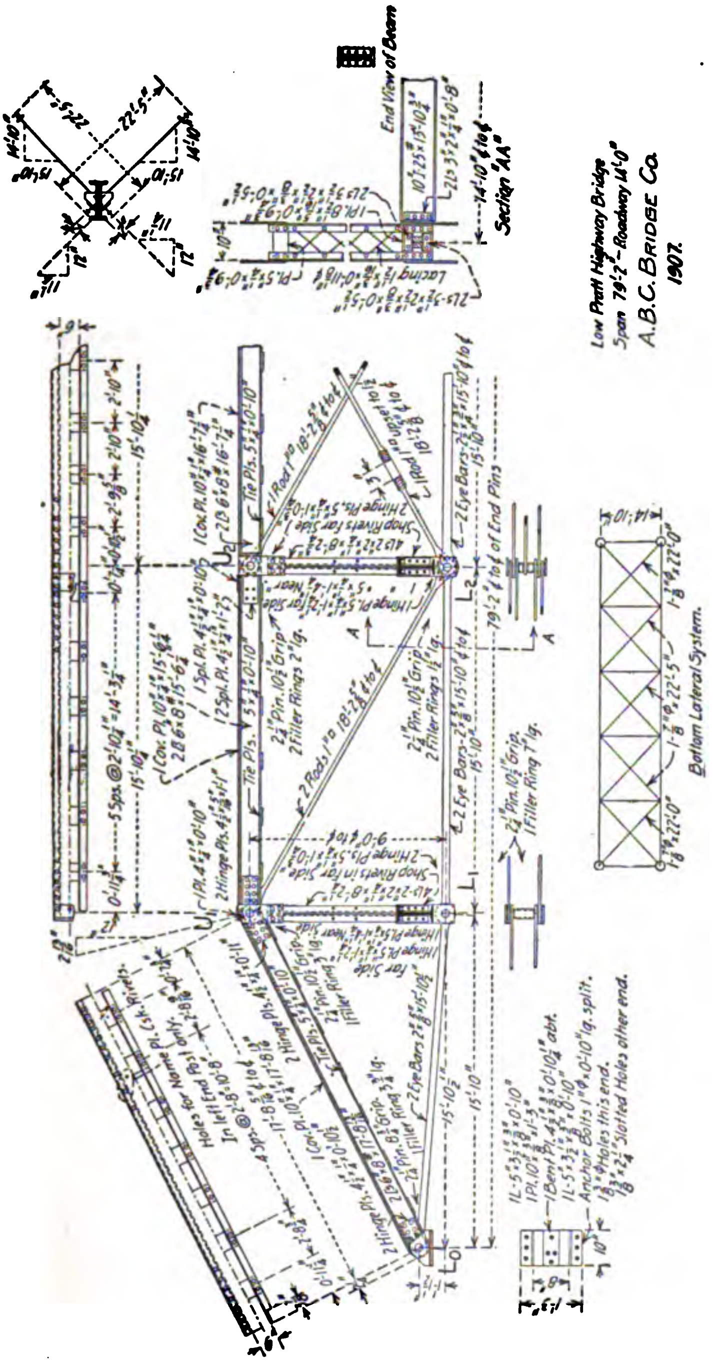



FIG. 10. LOW FULL SLOPE "FISH-BELLIED" PRATT TRUSS HIGHWAY BRIDGE.

## THE DESIGN OF A 70 FT. SPAN LOW RIVETED WARREN TRUSS BRIDGE.

1. **General Description of Bridge.**—This is to be a low truss bridge with riveted Warren trusses. The floor is to be composed of a reinforced concrete slab supported on I-beam joists. Provision will be made for a wearing surface weighing 50 lb. per sq. ft. of roadway. The floorbeams will be riveted to the posts above the lower chords to hold the trusses in place as rigidly as possible.

2. **LOADS.**—**Dead Load.**—The dead load consists of the weight of the reinforced concrete floor slab at 150 lb. per cu. ft., the wearing surface, joists, floorbeams, trusses and lateral bracing.

**Live Load.**—This bridge will be designed for Class $D_2$ loading which provides for a 15-ton concentrated load or a uniform load of 100 lb. per sq. ft. of roadway for the floor and its supports. The live load for the trusses is given in Table I of the specifications as 80 lb. per sq. ft. of roadway.

**Impact.**—The specifications provide for an allowance for impact of 30 per cent of the live load for the floor and its supports, and of

$$\frac{100}{L + 300} = \frac{100}{70 + 300} = 27 \text{ per cent}$$

for the truss members.

**Wind Load.**—The specifications require that the lateral bracing be designed for a moving wind load of 300 lb. per lineal foot of bridge.

3. **DIMENSIONS.**—Span, 70′ 0″ c. to c. of bearings; panel length, 14′ 0″; width of roadway, 16′ 0″, spacing of trusses, 17′ 3″ c. to c. about.

4. **Depth of truss,** ½ of panel length = 7′ 0″ c. to c. of chords. Minimum depth allowed by the specifications = $\frac{1}{10} \times 70$ = 7′ 0″.

5. **DESIGN OF FLOOR SYSTEM.**—The methods used in the design of the floor system of this bridge are given in Chapter X, and will not be repeated.

The slab, joist and floorbeams required may be found in Table I, and Fig. 9, Chapter X.

The following floor system will be used.

*Slab.*—Total thickness 6 in. Depth to center of steel 5 in. Reinforcement at right angles to joist; ½ in. square rods 6½ in. c. to c. Reinforcement parallel to joists; two ½ in. square bars between adjacent joists.

*Joists,* 9 in. I's @ 21 lb. spaced about 2 ft. 3 in. c. to c.

*Intermediate floorbeams,* 20 in. I's @ 65 lb.

*End floorbeams,* 20 in. I's @ 65 lb.

*Weight of floor system per panel*

| | | | |
|---|---|---|---|
| Wearing surface, | 50 × 14 × 16 | = | 11,200 lb. |
| Slab, | 75 × 14 × 16 | = | 16,800 " |
| Joists, | 8 × 21 × 14 | = | 2,400 " |
| Floorbeam, | 17 × 65 | = | 1,100 " |
| Total per panel | | = | 31,500 lb. |

Total weight of floor system = 31,500 × 5 = 157,500 lb. for entire bridge.

In order to calculate the live load floorbeam reaction it is necessary to calculate, first, the maximum load that can come on a floorbeam and second, the maximum reaction that can occur due to this load. From (*a*) Fig. 11, the greatest live load that can come on the floorbeam will be 10 + 4 × 5/14 = 11.44 tons. From (*b*) Fig. 11, it will be seen that the maximum reaction will occur with the wheels as shown. (The truck is 10 ft. wide.)

$$R_1 = \frac{11.63 \times 11.44}{17.25} \times 2{,}000 = 15{,}300 \text{ lb.}$$

Allowing 30 per cent for impact, the reaction will be 1.30 × 15,300 = 20,000 lb.

| | | |
|---|---|---|
| D. L. floorbeam reaction, intermediate | 15,800 lb. | End 7,900 lb. |
| L. L. and Impact floorbeam reaction, intermediate | 20,000 " | End 20,000 " |
| Total floorbeam reaction, intermediate | 35,800 lb. | End 27,900 lb. |

6. **STRESSES IN TRUSSES AND LATERAL SYSTEM.** Joint Load.—*Dead Load.*—

| | |
|---|---|
| Steel exclusive of floor system | 20,000 lb. |
| Floor system and wearing surface | 157,500 " |
| Total | 177,500 lb. |

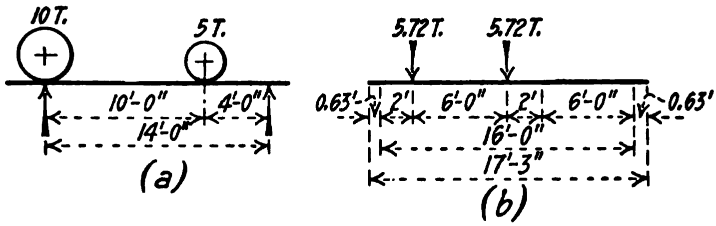



FIG. 11.

Dead joint load = 177,500 ÷ 10 = 17,750 lb.

*Live Load.*—80 × 16 × 14 ÷ 2 = 9,000 lb. per joint.

*Impact.*—0.27 × 80 × 16 × 14 ÷ 2 = 2,420 lb. per joint.

*Wind Load.*—300 × 14 = 4,200 lb. per joint, on lower chord only.

7. **Dead Load Stresses.**—The dead load stresses as calculated by the method of coefficients are shown on Fig. 12. The entire dead joint load was considered as applied at the lower chord joints.

8. **Live Load and Impact Stresses.**—The live load web stresses as calculated by the method of coefficients are shown in Fig. 12. The maximum chord stresses occur when the bridge is fully loaded and are obtained by multiplying the corresponding dead load stresses by 9,000 ÷ 17,750 = 0.507.

The impact stresses are determined by multiplying the live load stresses by the impact factor 0.27, and are given in Fig. 12.

9. **Wind Load Stresses.**—The wind load stresses as calculated by the method of coefficients are shown in Fig. 12. The wind load was considered as moving so the maximum stresses in the laterals will occur with the longer segment loaded and in the chords with all joints loaded. The total wind load was considered as concentrated at the joints of the windward truss. The total shear in any panel was assumed as taken by the member which would carry it in tension. The effect of the solid floor was not considered, although after the bridge is completed the laterals will not be necessary as far as wind stresses are concerned.

10. **Stress Sheet.**—Before proceeding with the design of members the stresses due to the various loadings will be collected on the stress sheet, Fig. 13. As soon as the size of each member is determined it will be recorded on the stress sheet.

11. **DESIGN OF MEMBERS.**—The upper chord and end-post sections will be made of two channels, flanges turned out, with a cover plate on top and lacing on the under side. The section should be made wide to provide lateral rigidity and the top chord should be supported horizontally by brackets at each floorbeam. The width of cover plates given by the following formula represents conservative practice, $b = L/10 + 6$ in.; where $b$ = minimum width in inches, and $L$ = length of span in feet. The width of the cover plate should preferably be in even inches.

The lower chord sections will be made of two angles, battened, placed outside of gusset plates with legs turned out where sufficient area can be provided. Where a greater area is required than

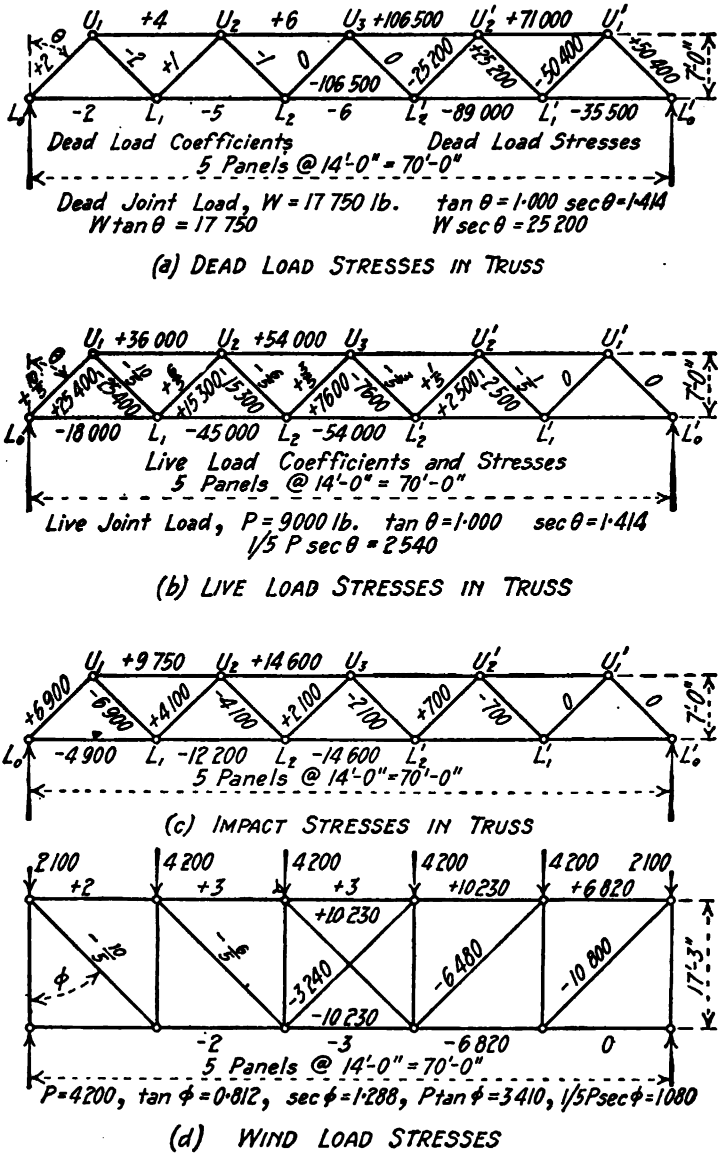



FIG. 12.

can be provided by two angles of reasonable size, four angles will be used, two with legs turned out placed outside of gusset plate, and two with legs turned in, placed inside of gusset plates.

The inclined web members will be made of two angles battened with legs turned in and placed inside of gusset plates.

The vertical posts do not carry any calculated stress, but serve to support the top chord against bending in a vertical plane and provide for the floorbeam connection. These members will be made of four angles and a web plate placed between the gusset plates, and entering the top

TABLE VII.

DATA FOR DESIGN OF 70 FT. SPAN LOW TRUSS STEEL BRIDGE.

| 1. | | 2. | 3. | 4. | 5. | 6. | 7. | 8. | 9. | 10. | 11. | 12. | 13. | 14. | 15. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Location. | Stress. | Member. | $D+L+I.$ | $W.$ | $\frac{W}{D+L+I}$ | Length, $l.$ | Radius of Gyration $r.$ | $\frac{l}{r}.$ | Allow. Unit Stress. | Req. Gross Area. | Req. Net Area. | Section. | Outstanding Leg. | Actual Gross Area. | Actual Net Area. |
| | | | Lb. | Lb. | | In. | In. | | Lb./In.² | In.² | In.² | | | In.² | In.² |
| Truss Members | Tension | $L_0L_1$ | 58,400 | 6,820 | .12 | 168 | 1.61 | 104 | 16,000 | — | 3.65 | 2 ∠s 5 × 3½ × 5/16 | s | 5.12 | 4.02 |
| | | $L_1L_2$ | 146,200 | 10,230 | .07 | 168 | 1.60 | 105 | 16,000 | — | 9.14 | 2 ∠s 5 × 3½ × 5/16, 2 ∠s 5 × 3½ × 7/16 | s | 12.18 | 9.54 |
| | | $L_2L_2'$ | 175,100 | 10,230 | .06 | 168 | 1.59 | 106 | 16,000 | — | 10.95 | 4 ∠s 5 × 3½ × 7/16 | s | 14.12 | 11.04 |
| | | $U_1L_1$ | 82,700 | 0 | — | 84 | 1.59 | 53 | 16,000 | — | 5.17 | 2 ∠s 5 × 3½ × 7/16 | s | 7.06 | 5.52 |
| | | $U_2L_2$ | 44,600 | 0 | — | 84 | 1.61 | 52 | 16,000 | — | 2.79 | 2 ∠s 5 × 3½ × 5/16 | s | 5.12 | 4.02 |
| | | $U_3L_2$(1) | 14,550 | 0 | — | 84 | 1.12 | 75 | 16,000 | — | 0.91 | 2 ∠s 3½ × 2½ × ¼ | s | 2.88 | 2.44 |
| | Compression | $L_0U_1$ | 82,700 | 0 | — | 119 | 3.16 | 38 | 13,370 | 6.18(4) | — | 2 [s 8″ × 11¼ lb., 1 pl. 14 × 5/16(4) | — | 11.08 | — |
| | | $U_1U_2$ | 116,800 | 0 | — | 168(6) | 4.57(6) | 36(6) | 13,500 | 8.65 | — | 2 [s 8″ × 11¼ lb., 1 pl. 14 × 5/16 | — | 11.08 | — |
| | | $U_2U_3$ | 175,100 | 0 | — | 168(6) | 4.62(6) | 36(6) | 13,450 | 13.02 | — | 2 [s 8″ × 16.25, lb., 1 pl. 14 × 5/16 | — | 13.94 | — |
| | | $L_1U_2$ | 44,600 | 0 | — | 119 | 1.61 | 74 | 10,820 | 4.12 | — | 2 ∠s 5 × 3½ × 5/16 | s | 5.12 | — |
| | | $U_3L_2$(1) | 14,550 | 0 | — | 119 | 1.12 | 106 | 8,560 | 1.70 | — | 2 ∠s 3½ × 2½ × ¼ | s | 2.88(5) | — |
| | | Posts | 0 | 0 | — | 84 | | | — | — | — | 4 ∠s 3 × 2½ × ¼ | | | |
| Laterals | Tension | $L_0L_1$ | — | 10,800 | — | 133(3) | 0.77 | 173 | 20,000 | — | 0.54 | 1 ∠ 2½ × 2½ × ¼ | — | 0.89(2) | 0.73(2) |
| | | $L_1L_2$ | — | 6,480 | — | 133 | 0.77 | 173 | 20,000 | — | 0.33 | 1 ∠ 2½ × 2½ × ¼ | — | 0.89 | 0.73 |
| | | $L_2L_2'$ | — | 3,240 | — | 133 | 0.77 | 173 | 20,000 | — | 0.16 | 1 ∠ 2½ × 2½ × ¼ | — | 0.89 | 0.73 |

(1) Subjected to both tension and compression during passage of one load so stress = 9,700 + ½ × 9,700 = 14,550 lb.

(2) Single angle member connected by one leg so effective area = 75 per cent of computed area.

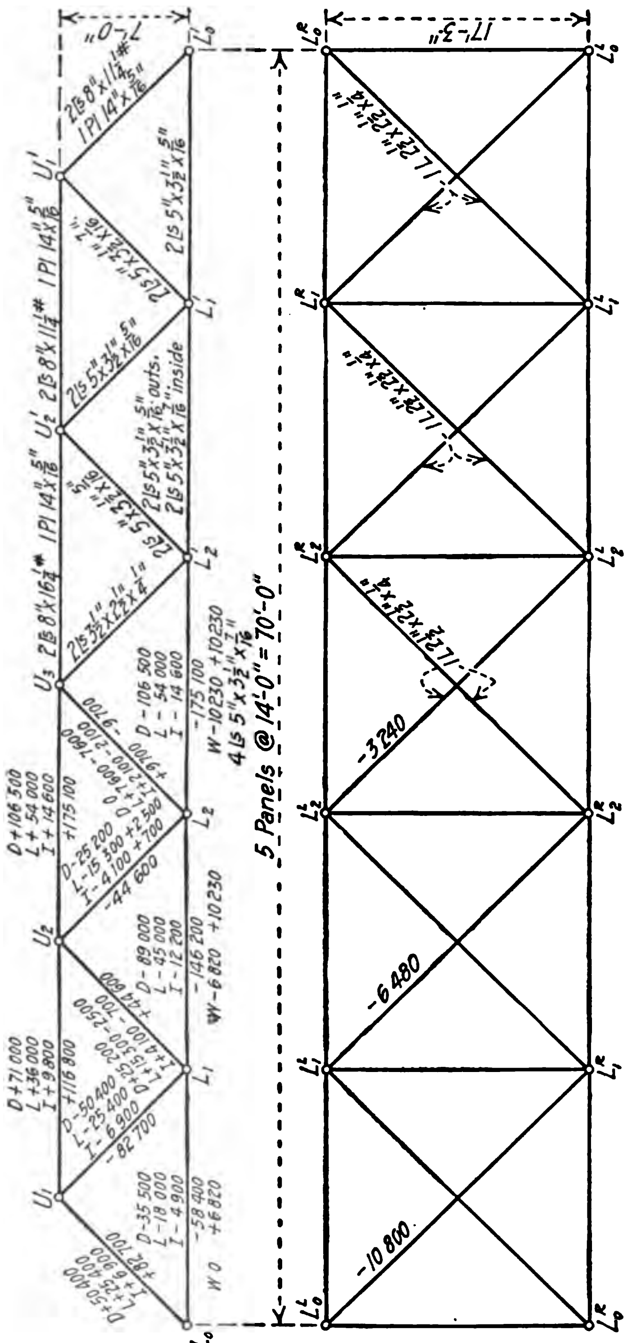
(3) Connected at intersection so length is taken as ½ of total length.

(4) See discussion on selection of channels given at the beginning of the design of joints.

(5) 2 angles 2½ × 2½ × ¼ give $l/r$ = 119/.77 = 153. Allowable = 125, so section cannot be reduced.

(6) The $l/r$ for bending in a horizontal plane and for bending in a vertical plane are both calculated, and the values giving the maximum recorded.

14




STRESS SHEET

FIG. 13.

chord section. The size of the angles and plate will be determined principally by convenience in detailing the structure. The lateral system will be composed of single angle members.

The design of the members is given in the Table VII, but for the sake of explanation the complete calculations for one of each type of member will be given. The size of most of the truss members will depend upon the width of the top chord sections so the top chord carrying the largest stress will be designed first.

**12. Design of the Top Chord $U_2U_3$.**

| | | |
|---|---|---|
| Dead load stress, | 106,500 lb. compression | |
| Live load stress, | 54,000 " compression | |
| Impact stress, | 14,600 " compression | |
| Total | 175,100 lb. compression | (Record in Col. 3) |
| Wind load stress, | 0 | (Record in Col. 4) |
| Length for bending in a horizontal plane | 168 in. | (Record in Col. 6) |
| Length for bending in a vertical plane, | 84 in. | (Record in Col. 6) |

The width of the cover plate should be at least

$$b = L/10 + 6 \text{ in.} = 70/10 + 6 = 13 \text{ in.}$$

and taking the width in even inches, 14 in. will be used.

Since the member is supported at the center against bending in a vertical plane, bending in a horizontal plane will probably control. Approximate radii of gyration of structural sections are given in Table 43, Appendix III. The approximate radius of gyration for an axis perpendicular to the cover plate is 0.32b for this type of section or

$$r_B = 0.32 \times 14 = 4.48 \text{ in.}$$

Using this radius of gyration and the corresponding length, the approximate allowable unit stress is

$$S = 16{,}000 - 70 \times 168/4.48 = 13{,}380 \text{ lb. per sq. in.}$$

The approximate area required is

$$A = P/S = 175{,}100/13{,}380 = 13.07 \text{ sq. in.}$$

The gage lines of the cover plate will not be farther apart than $14 - 2 \times 1\frac{1}{4} = 11.5$ in. allowing an edge distance of $1\frac{1}{4}$ in. on each side. The thickness of the cover plate must be at least $1/40 \times 11.5 = 0.288$ in. so a $\frac{5}{16}$ in. plate must be used. The minimum cover plates should be used whenever possible. Deducting the area of the 14 in. $\times$ $\frac{5}{16}$ in. cover plate from the total area required, the area to be provided by the two channels is

$$13.07 - 4.38 = 8.69 \text{ sq. in.}$$

or 4.35 sq. in. for each channel.

Since the same general dimensions must be used on all the members of the top chord and end-post the other members must be kept in mind in choosing the section. By referring to the stress sheet it is seen that the stress in the end-post is less than one-half as great as the one under consideration, so a depth of channel should be chosen that will furnish much lighter weights than the one used for this member. Referring to a table of the properties of channels it is seen that an 8 in. channel @ 16.25 lb., a 7 in. channel @ 14.75 lb. and a 6 in. channel @ 15.5 lb. will all provide sufficient area for this case and also allow considerable reduction in area. The 8 in. channel @ 16.25 lb. will be tried.

The width of flange of a 8 in. channel @ 16.25 lb. is $2\frac{1}{2}$ in., and the gage is $1\frac{1}{2}$ in., leaving 1 in. of flange projecting beyond the gage. The rivet line of the cover plate should therefore not be less than 1 in. from the edge of the plate. The maximum rivet allowed in the flange of a 8 in. channel is $\frac{3}{4}$ in., and the minimum edge distance for a $\frac{3}{4}$ in. rivet is $1\frac{1}{4}$ in. An edge distance of $1\frac{1}{4}$ in.

will be used making the rivet lines of the plate $14 - 2\frac{1}{2} = 11\frac{1}{2}$ inches apart, and the distance back to back of channels $11\frac{1}{2} - 3 = 8\frac{1}{2}$ in.

13. **Details of Design of $U_2U_3$.**—The properties of the section composed of 2 channels 8 in. @ 16.25 lb. placed $8\frac{1}{2}$ in. back to back and a cover plate 14 in. $\times \frac{5}{16}$ in. will now be calculated. See Fig. 14.

*Area.*—

$$\begin{array}{lll} \text{Channels } 2 \times 4.78 & = & 9.56 \text{ sq. in.} \\ \text{Plate} \quad 4.38 & = & \underline{4.38} \text{ " "} \\ \quad\text{Total} & = & 13.94 \text{ sq. in. (Col. 14)} \end{array}$$

*Centroid.*—The distance from the axis $M-M$ to the centroid of the section is found by taking moments about axis $M-M$ and dividing by the area

$$\begin{array}{lll} \text{Moment of channels} & & 0 \\ \text{Moment of plate} & 4.16 \times 4.38 = & \underline{18.22} \\ \quad\text{Total} & = & 18.22 \end{array}$$

$$e = 18.22/13.94 = 1.31 \text{ in.}$$

*Moment of Inertia Axis A–A.*—The moment of inertia about the axis $M-M$ is first determined. The value of $I_A$ is found by subtracting from $I_M$, the quantity $Ae^2$.

$$\begin{array}{lll} \text{Channels, } 2 \times I & = 2 \quad \times 39.9 = & 79.8 \text{ in.}^4 \\ \text{Plate, area} \times \text{square of distance from center to } M-M & = 4.38 \times 4.16^2 = & \underline{75.8} \text{ "} \\ \quad\text{Total } I_M & = & 155.6 \text{ in.}^4 \end{array}$$

$$I_A = I_M - Ae^2 = 155.6 - 13.94 \times 1.31^2 = 131.7 \text{ in.}^4$$

*Moment of Inertia axis B–B.*—

$$\begin{array}{lll} \text{Channels, } 2(1.78 + 4.78 \times 4.81^2) & = & 224.76 \text{ in}^4. \\ \text{Plate, } \frac{1}{12} b\cdot d^3 = \frac{1}{12} \times \frac{5}{16} \times 14^3 & = & \underline{71.50} \text{ "} \\ \quad\text{Total} & = & 296.26 \text{ in}^4. \end{array}$$

*Radii of gyration.*

$$\text{Axis } A-A,\ r_A = \sqrt{\frac{I_A}{A}} = \sqrt{\frac{131.7}{13.94}} = 3.07 \text{ in. (Col. 7)}$$

$$\text{Axis } B-B,\ r_B = \sqrt{\frac{I_B}{A}} = \sqrt{\frac{296.26}{13.94}} = 4.62 \text{ in. (Col. 7)}$$

Value of $\frac{l}{r}$ for bending in a horizontal plane, $\frac{168}{4.62} = 36.4$ (Col. 8)

Value of $\frac{l}{r}$ for bending in a vertical plane, $\frac{84}{3.07} = 27.4$ (Col. 8)

Maximum $l/r$ allowed for main compression members is 125 so this section is well below the limit. The maximum $l/r$ occurs for bending in a horizontal plane and is 36.4. The allowable unit stress, $S = 16{,}000 - 70l/r = 16{,}000 - 70 \times 36.4 = 13{,}450$ lb. per sq. in. (Col. 9)

$$A = P/S = \frac{175{,}100}{13{,}450} = 13.02 \text{ sq. in. (Col. 10)}$$

Actual area is 13.94 sq. in., so the section assumed is satisfactory as sufficient area is provided and no reduction is possible.

14. **Design of Lower Chord $L_2L_2'$.**

The lower chord carrying the maximum stress is $L_2L_2'$ so this member should be designed first.

| | | |
|---|---|---|
| Dead load stress, | 106,500 lb. tension | |
| Live load stress, | 54,000 lb. tension | |
| Impact, | 14,600 lb. tension | |
| Total | = 175,100 lb. tension | (Col. 3) |
| Wind, 10,230 lb. tension or compression (Col. 4) | | |

The wind load tension is less than the live load tension, the wind load tension is less than 25 per cent of the sum of dead load, live load, and impact, and the wind load compression is less than the dead load tension, so the wind stress may be neglected (Col. 5).

The net area required for this member is

$$A = \frac{P}{S} = \frac{175,100}{16,000} = 10.95 \text{ sq. in. (Col. 11)}$$

Deducting one less holes than there are gage lines on the angles and considering a $\frac{7}{8}$ in. hole for a $\frac{3}{4}$ in. rivet, the following sections could be used:

$$2 \text{ angles } 6'' \times 6'' \times \tfrac{5}{8}'', \text{ net area } = 2(7.11 - 2 \times 0.55) = 12.02 \text{ sq. in.}$$
$$2 \text{ angles } 6'' \times 4'' \times \tfrac{3}{4}'', \text{ net area } = 2(6.94 - 2 \times 0.66) = 11.24 \text{ sq. in.}$$

No material thicker than $\frac{3}{4}$ in. should be used on account of the difficulty in punching, so if two angles are used one of the above sections must be adopted. A more satisfactory section can probably be obtained by using four angles. The clearance between the gusset plates must be calculated before the angles which are to come between the plates can be determined. This distance will be made the same for both upper and lower chords. The thickness of the gusset plates will be determined later under the design of joints, but will not exceed $\frac{5}{8}$ in. The distance b. to b. of channels of the top chord is $8\frac{1}{2}$ in., so the clear distance between gusset plates is at least $8\frac{1}{2} - 2 \times \frac{5}{8} = 7\frac{1}{4}$ in. To provide for clearance and overrun of angles, and allow drainage the turned in legs of the angles should not be larger than $3\frac{1}{2}$ in.

The net area provided by four angles $5'' \times 3\frac{1}{2}'' \times \frac{7}{16}''$ is

$$4(3.53 - 2 \times 0.38) = 11.08 \text{ sq. in.}$$

The required net area is 10.95 sq. in. so this section will be adopted if the ratio of $l/r$ does not exceed 200. The 5 in. leg will be placed vertical. The radius of gyration about the horizontal axis is 1.59 in. (Col. 7), and the length is 168 in. (Col. 6), making $l/r = 168/1.59 = 106$ which is satisfactory (Col. 8).

The other members are designed in a similar manner, the results of the calculations being given in Table VII.

15. **Design of Joints.**—All joints will be designed to develop the full strength of the members and not simply the calculated stress.

The gusset plates will be made at least thick enough to develop in bearing the strength of the rivets in single shear. Referring to Table 33, Appendix III, this thickness is seen to be $\frac{7}{16}$ in. The plates must be of sufficient size to contain the necessary rivets and to carry the stresses transmitted from the members.

All rivets will be made $\frac{3}{4}$ in. in diameter except those in the flanges of channels whose depth is less than 8 in. (See Table 17, Appendix III, maximum rivets for channel flanges.) The use of lug angles will be avoided wherever possible.

The allowable values for rivets in shear and bearing are given in Table 33, Appendix III.

The arrangement of the members at the joints is shown on the general drawings, Fig. 16. The gage lines of angles are placed on the center line of the truss. Where an angle has two gage lines the one nearest the back is used. The centers of gravity of the top chords and end-posts are placed on the center line of the truss. The size of gusset plates is usually determined by the space required for the rivets necessary to connect the members to the plate. Except in extreme

cases the size of gusset plates required by the rivets will be sufficient for strength. To secure uniformity of stress the rivets should be symmetrically spaced.

The channels to be used for the top chord and end-post are often determined by the thickness of web, for bearing on the web may determine the number of rivets required at the joint $U_1$, and the number of rivets determines the size of plate. This size must not be excessive. In the design of this bridge 7 in. channels @ 9¾ lb. were found to be satisfactory for strength, but the thickness of web is only 0.21 in. This required $\frac{13,480 \times 10.08}{.75 \times .21 \times 24,000} = 36$ shop rivets for the member $U_1U_2$. Only two lines of rivets can be placed in a 7 in. channel, so the plate required was excessively large. By using 7 in. channels @ 12¼ lb. the thickness of web is increased to 0.318 in., which would require $\frac{13,520 \times 11.58}{5,300} = 30$ shop rivets. To avoid alterations in the design it is desirable to design and detail the joint $U_1$ before the top chord channels are finally adopted. In this design 8 in. channels were adopted because three rows of rivets could be used.

16. **Joint $U_1$.**—This joint should be designed first because the size of channels may be determined by the number of rivets required at the end of $U_1U_2$. It would be advantageous to design this joint before the top chord and end-post channels are selected.

A diagram showing the stresses in members fastened to the gusset plate at $U_1$, is given in Fig. 14.

The gusset plates will be shop riveted to the member $U_1U_2$ and field riveted to all other members.

Bearing controls the number of rivets in $L_0U_1$ at $U_1$. The number of field rivets required will be $\frac{148,200}{.75 \times .22 \times 20,000} = 45$ or 23 on each side.

The number of shop rivets in $U_1U_2$ at joint $U_1$ is determined by bearing, and is

$$\frac{149,600}{.75 \times .22 \times 24,000} = 38 \text{ or } 19 \text{ on each side.}$$

The number of field rivets in $U_1L_1$ at joint $U_1$ is determined by shear, and is $\frac{88,300}{4,420} = 20$ or 10 on each side.

17. **Joint $L_0$.**—Cast iron shoes will be used at the fixed end, and cast iron rockers at the expansion end. The details of the shoes and rockers are shown in Fig. 15.

The pin at $L_0$ should be made as large as the channels of $L_0U_1$ will permit even though a smaller pin would safely carry the stresses. The sizes in most common use are 3, 3½ and 4 inches in diameter. The channels of the end-post are 8 × 11¼ lb. A pin 3 in. in diameter will be used if the following investigation shows it to have sufficient strength.

For the detail here used the forces acting on the pin are all vertical and have a magnitude equal to one-half of the maximum pedestal reaction. This maximum will occur with the bridge fully loaded and will equal one-half of the sum of the dead load, live load and impact joint loads, multiplied by the number of panels or

$$R = \tfrac{1}{2}(17,750 + 9,000 + 0.27 \times 9,000)5 = 73,000 \text{ lb.}$$

The arrangement at the joint is shown in Fig. 14.

The thickness of the gusset plate is determined by the bearing area required, and is

$$t = \frac{36,500}{3 \times 24,000} = 0.51 \text{ in.}$$

The thickness of the channel web is 0.22 in., making the required thickness of the gusset plate equal to $0.51 - 0.22 = 0.29$ in. A $\frac{5}{16}$-in. plate would satisfy this requirement, but a ⅜-in. plate will be used.

The maximum bending moment in the pin is

$$M = 36,500 \times 1.71 = 62,400 \text{ in. lb.}$$

The maximum shear on the pin is

$$V = 36{,}500 \text{ lb.}$$

The diameter required by bending moment is

$$d = \left(\frac{32M}{\pi \cdot f}\right)^{\frac{1}{3}} = 2.17\left(\frac{M}{f}\right)^{\frac{1}{3}} = 2.17\left(\frac{62{,}400}{24{,}000}\right)^{\frac{1}{3}} = 2.98 \text{ in.}$$

The diameter required by shear is

$$d = \sqrt{\frac{4V}{\pi \cdot f_s}} = 1.13\sqrt{\frac{V}{f_s}} = 1.13\sqrt{\frac{36.500}{12{,}000}} = 1.97 \text{ in.}$$

The 3-in. pin is therefore satisfactory and will be used. For bending moments on pins see Table 27, Appendix III.

A diagram showing the members fastened to the gusset plates at $L_0$ is given in Fig. 14. The stresses shown were determined by multiplying the allowable unit stresses by the gross area for compression members and by the net area for tension members. These values are taken from Table VII. The end-post will be shop riveted to the gusset plate, and the lower chord and floorbeam will be field riveted.

The number of ¾-in. shop rivets required in $L_0U_1$ at joint $L_0$ is determined by bearing on the web of the channels, and is

$$\frac{148{,}200}{.75 \times .22 \times 24{,}000} = 38 \text{ or } 19 \text{ on each side.}$$

The number of ¾ in. field rivets required in $L_0L_1$ at joint $L_0$ is determined by single shear, and is

$$\frac{64{,}300}{4{,}420} = 15 \text{ or } 8 \text{ on each side.}$$

The number of ¾-in. field rivets required between the connection angles of the floorbeam and the gusset plate is

$$\frac{27{,}900}{4{,}420} = 7 \text{ rivets.}$$

The section area of gusset plate required to carry the stress in $L_0L_1$ is equal to the net area of the member, or 4.02 sq. in. With two gusset plates each ⅜ in. thick the effective net width must be

$$\frac{4.02}{0.75} = 5.4 \text{ in.}$$

It is evident from the detail drawings that sufficient area has been provided.

The number of field rivets required for all bottom laterals is

$$\frac{0.73 \times 20{,}000}{1.25 \times 3{,}750} = 3 \text{ rivets.}$$

**18. Joint $L_1$.**—A diagram showing the members fastened to the gusset plates at $L_1$ is given in Fig. 14. The stresses shown are determined as explained under the design of the joint $L_0$.

The gusset plates will be shop riveted to $L_1L_2$ and field riveted to all other members.

The number of ¾-in. field rivets required by $U_1L_1$ is determined by single shear, and is

$$\frac{89{,}300}{4{,}420} = 20 \text{ or } 10 \text{ on each side.}$$

The number of ¾-in. field rivets required by $L_1U_2$ is determined by single shear, and is

$$\frac{55{,}400}{4{,}420} = 13 \text{ or } 7 \text{ on each side.}$$

The number of $\frac{3}{4}$-in. field rivets required by $L_0L_1$ is determined by single shear, and is

$$\frac{64,300}{4,420} = 15 \text{ or } 8 \text{ on each side.}$$

The bottom lateral plate is field connected to this member and will assist in transmitting stress from $L_0L_1$ to $L_1L_2$. The maximum stress which can be carried in this way is equal to the stress value of the legs riveted to the plate. This stress value should be determined for $L_0L_1$ and $L_1L_2$ and the lesser value used. It is evident that $L_0L_1$ controls. The portion of the net area furnished by the attached legs is $4.02 \times 3.5 \div 8.5 = 1.66$ sq. in. which carries a stress of $1.66 \times 16,000 = 26,500$ lb. The number of rivets required to develop this stress in single shear is

$$\frac{26,500}{4,420} = 6 \text{ rivets}$$

so the number of rivets between the lateral plate and the angles of $L_0L_1$ is 3 for each angle. This enables the rivets in the gusset plate to be reduced to $8 - 3 = 5$ on each side. If more rivets are used in the lateral plate only three can be counted on each side in reducing the number of rivets in the gusset plates.

The number of $\frac{3}{4}$-in. shop rivets required for $L_1L_2$ is determined by bearing on the gusset plate. A $\frac{3}{8}$-in. gusset plate will be used so the number required is

$$\frac{153,000}{6,750} = 23 \text{ or } 12 \text{ on each side.}$$

This number can be reduced by 3 for each side as calculated under $L_0L_1$ making 9 on each side, not considering the difference in value of field and shop rivets in this case.

The number of field rivets required at the ends of the bottom laterals is 3 as calculated under the joint $L_0$. This number will be used throughout the bridge.

The number of $\frac{3}{4}$-in. field rivets required for the floorbeam connection is the same for all intermediate floorbeams and is

$$\frac{35,800}{4,420} = 8 \text{ rivets.}$$

19. **Joint $U_2$.**—The arrangement of the members connected to the gusset plates at $U_2$ is shown in Fig. 14. The stresses are calculated as explained under the joint $L_0$.

The member $U_2U_3$ will be shop riveted to the gusset plate. All other members will be field riveted.

When a member is spliced at a point away from the joint and the abutting ends are milled, the splice is usually designed to carry from 50 to 75 per cent of the total stress. If the ends are not milled the full stress should be used. In the bridge under consideration the splice is placed at the joint. If the abutting ends are milled the member carrying the smaller stress would be designed to carry 50 to 75 per cent of this stress, while the member carrying the larger stress would be designed to carry 50 to 75 per cent of the smaller stress plus the total value of the difference in the two stresses.

If the splice is at the joint and the members are not milled each connection should be designed to carry full stress in the member. As the stresses are not large enough to require excessive gusset plates if the connections are designed for the full stress, and considering the uncertainty of the bearing, this method will be adopted and no reliance will be placed on the milled ends.

The number of $\frac{3}{4}$-in. field rivets required for $U_1U_2$ is determined by bearing, and is

$$\frac{149,600}{.75 \times .22 \times 20,000} = 46 \text{ or } 23 \text{ on each side.}$$

The 14 in. $\times$ $\frac{5}{16}$ in. cover plate should be spliced for its full stress value of $4.38 \times 13,480 = 59,000$ lb. The number of field rivets required at one side of the splice is

$$\frac{59,000}{4,420} = 14 \text{ rivets.}$$

so the number of rivets required in the gusset plates can be reduced to $23 - 7 = 16$ on each side.

The number of $\frac{3}{4}$ in. shop rivets required for $U_2U_3$ is determined by single shear, and is

$$\frac{187,600}{5,300} = 36 \text{ or } 18 \text{ on one side.}$$

The number of $\frac{3}{4}$ in. shop rivets required to develop the cover plate is (field rivets were used)

$$\frac{59,000}{5,300} = 12 \text{ rivets}$$

so the number of rivets required in gusset plates can be reduced to $18 - 6 = 12$ on each side.

The number of $\frac{3}{4}$-in. field rivets required for $L_1U_2$ is determined by single shear, and is

$$\frac{55,400}{4,420} = 13 \text{ or } 7 \text{ on each side}$$

and for $U_2L_2$ is

$$\frac{64,300}{4,420} = 14 \text{ or } 7 \text{ on each side.}$$

**20. Joint $L_2$.**—The member $L_1L_2$ will be shop riveted to the gusset plate. All other members will be field riveted.

The number of $\frac{3}{4}$-in. shop rivets required in $L_1L_2$ is determined by bearing on the $\frac{3}{8}$-in. gusset plate. The number is

$$\frac{153,000}{6,750} = 23 \text{ or } 12 \text{ on each side.}$$

The bottom lateral plate is shop riveted to $L_1L_2$ and field riveted to $L_2L_3$. The stress value of the legs riveted to this plate can be transmitted through the plate. The stress value of $L_1L_2$ controls. The portion of the net area furnished by the legs is $9.54 \times 3.5 \div 8.5 = 3.92$ sq. in., which carries a stress of $3.92 \times 16,000 = 63,000$ lb. The number of $\frac{3}{4}$-in. shop rivets required to develop this stress in single shear is $63,000/5,300 = 12$ rivets, so that the number of rivets between the lateral plates and the angles of $L_1L_2$ is 3 for each angle. This enables the number of rivets in the gusset plate to be reduced to $12 - 6$ or 6 rivets on each side.

The number of $\frac{3}{4}$-in. field rivets required for $U_2L_2$ is determined by single shear, and is

$$\frac{64,300}{4,420} = 14 \text{ or } 7 \text{ on each side.}$$

The number of $\frac{3}{4}$-in. field rivets required for $L_2U_3$ is determined by bearing, and is

$$\frac{39,100}{3,750} = 10 \text{ or } 5 \text{ on each side.}$$

The number of $\frac{3}{4}$-in. field rivets required for $L_2L_2'$ is determined by bearing on the $\frac{3}{8}$-in. gusset plate, and is

$$\frac{176,500}{5,630} = 32 \text{ or } 16 \text{ on each side.}$$

This number may be reduced by 6 on each side as calculated under $L_1L_2$, making 10 on each side not considering the difference in value in shop and field rivets.

**21. Joint $U_3$.**—The number of $\frac{3}{4}$-in. field rivets required for $L_2U_3$ is determined by bearing, and is

$$\frac{39,100}{3,750} = 10 \text{ or } 5 \text{ on each side.}$$

The number of rivets at joint $U_3$ between the upper chord and gusset plate must be sufficient to transfer to the gusset plate the maximum difference in stress between the members $U_2U_3$ and $U_3U_2'$. This occurs when $U_3L_2'$ has its maximum stress. The difference is $\frac{6}{5}P \cdot \tan\theta$ + impact + difference in dead load stresses $= \frac{6}{5} \times 9,000 + .27 \times \frac{6}{5} \times 9,000 + 0 = 10,800 + 2,920 = 13,720$ (The stress in $U_2U_3$ and $U_3U_2'$ may be calculated by algebraic moments, with loads at $L_2'$ and $L_1'$. This serves as a check method.)

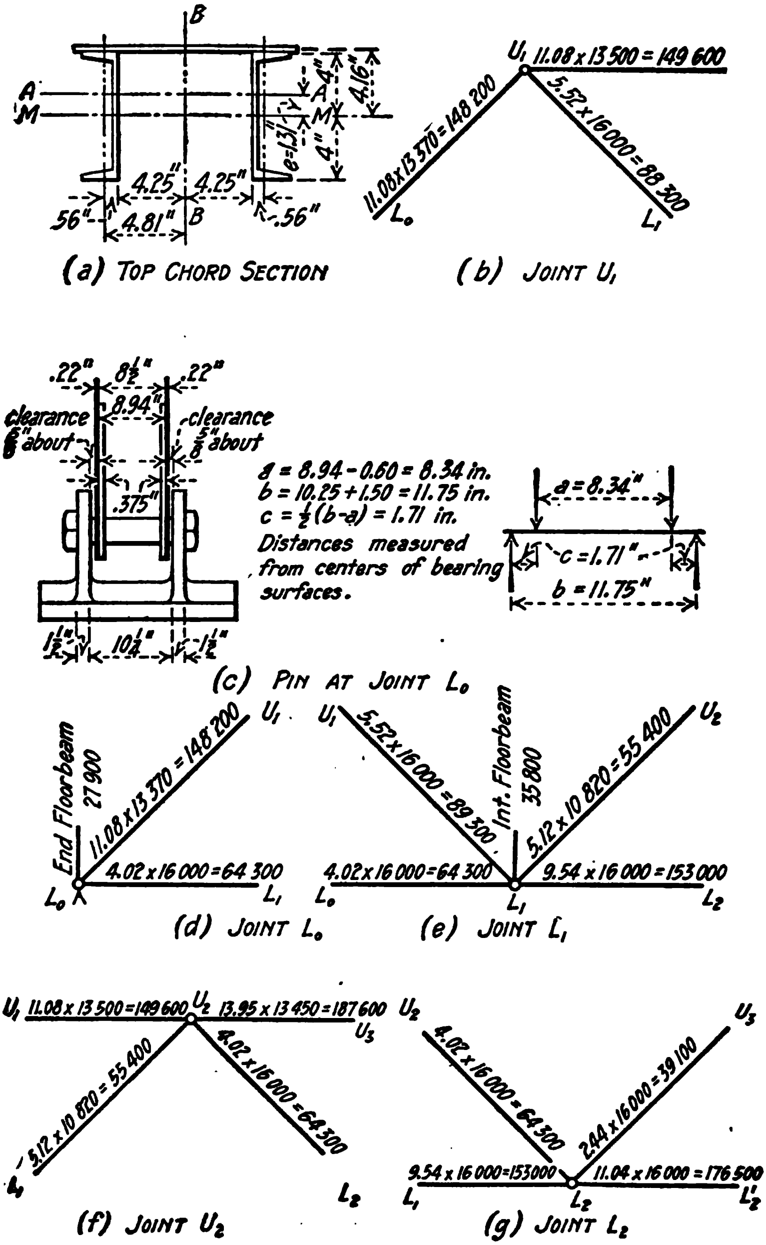

(a) TOP CHORD SECTION
(b) JOINT U1
U1 11.08 x 13 500 = 149 600
11.08 x 13 370 = 148 200
5.52 x 16 000 = 88 300
a = 8.94 - 0.60 = 8.34 in.
b = 10.25 + 1.50 = 11.75 in.
c = ½(b-a) = 1.71 in.
Distances measured from centers of bearing surfaces.
clearance ⅝" about
(c) PIN AT JOINT L0
End Floorbeam 27 900
4.02 x 16 000 = 64 300
(d) JOINT L0
Int. Floorbeam 35 800
5.52 x 16 000 = 89 300
5.12 x 10 820 = 55 400
9.54 x 16 000 = 153 000
(e) JOINT L1
13.95 x 13 450 = 187 600
(f) JOINT U2
2.44 x 16 000 = 39 100
11.04 x 16 000 = 176 500
(g) JOINT L2

FIG. 14.

The number of ¾-in. shop rivets required in $U_2U_2'$ at $U_2$ will be determined by single shear, and is

$$\frac{13{,}720}{5{,}300} = 3 \text{ rivets.}$$

In order to make a firm connection a greater number will be used.

**22. End Bearings.**—Rocker or roller bearings are required on spans of 70 ft. or more, so must be used for this bridge.

**23. Design of Cast Iron Rockers.** The type of rocker shown in Fig. 15 will be used. The maximum pedestal reaction as determined in par. 17, is 73,000 lb. The area of each masonry plate must be $\frac{73{,}000}{600} = 121$ sq. in., where 600 is the allowable bearing stress on concrete masonry. The length of the rocker will be taken as 23 inches so the bearing stress between the rocker and the plate is $p = \frac{73{,}000}{23} = 3{,}170$ lb. per lineal inch. The allowable bearing stress is 300 d pounds per lineal inch or $300 \times 18 = 5{,}400$ lb. per lin. in.

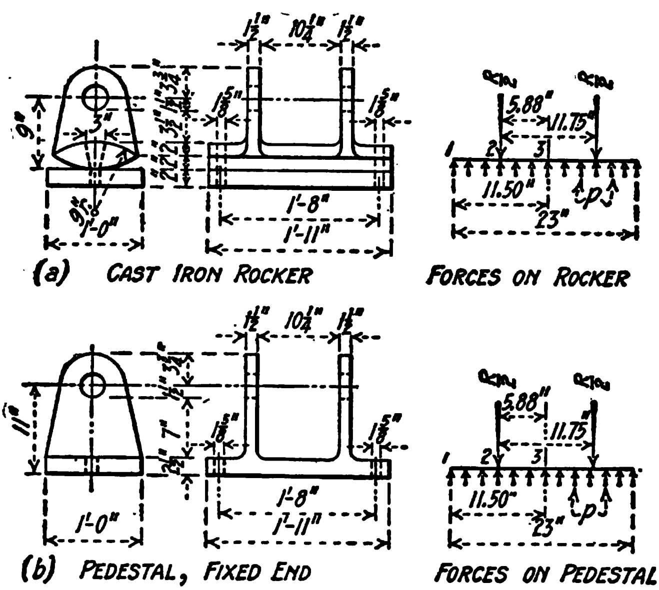



FIG. 15.

The forces acting on the rocker are shown in Fig. 15. The section will be investigated as a cantilever beam with an effective length of $11\frac{1}{2} - (5\frac{1}{8} + 1\frac{1}{2}) = 4\frac{7}{8}$ in.,

$$M_2 = 3{,}170 \times \frac{4.88^2}{2} = -\ 37{,}700 \text{ in.-lb.}$$

$$M_3 = -\ 3{,}170 \times \frac{11.50^2}{2} + 36{,}500 \times 5.88 = +\ 4{,}750 \text{ in.-lb.}$$

The moment of inertia of this section, graphically determined, is 28.88 in.$^4$, $c = 2$ in., so the largest bending stress is

$$S = \frac{M \cdot c}{I} = \frac{37{,}700 \times 2}{28.88} = 2{,}610 \text{ lb. per sq. in.}$$

which is safe, so a depth of 4 inches is sufficient for bending. The shear to the left of the upright is $4.88 \times 3{,}170 = 15{,}460$ lb. and to the right is $(11.50 - 5.13)3{,}170 - 36{,}500 = 16{,}300$ lb. The section area is 31.04 sq. in. so the largest average unit shear is

$$\frac{16{,}300}{31.04} = 525 \text{ lb. per sq. in.}$$

The depth of 4 inches is sufficient for bending and shear so will be used.

The thickness of the upright will be determined by the bearing area on the pin. Using an allowable bearing stress of 9,000 lb. per sq. in. for cast iron, for a 3-inch pin, we have $2 \times 3 \times t \times 9{,}000 = 73{,}000$ or $t = 1.35$ inches. $1\frac{1}{2}$ inches will be used.

The unsupported length of the upright is 5 inches and with a thickness of $1\frac{1}{2}$ inches, there will be no column action.

The type of pedestal shown in Fig. 15 will be used at the fixed end. The same bearing stress on the masonry exists here as at the expansion end. The forces acting are shown in Fig. 15. The maximum bending moment is $-37{,}700$ in.-lb. The moment of inertia of the section is $\frac{1}{12} \times 12 \times 2.5^3 = 15.6$ in$^4$.

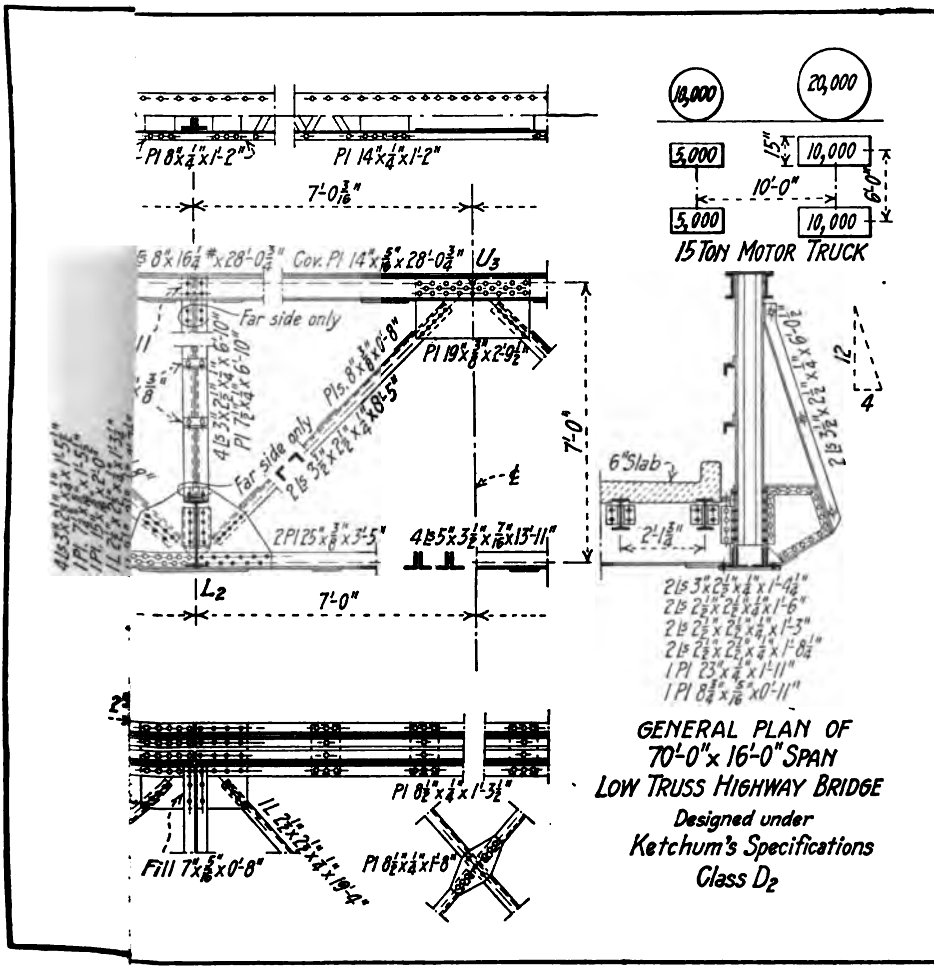
$$S = \frac{37{,}700 \times 1.25}{15.6} = 3{,}020 \text{ lb. per sq. in.}$$

The maximum shear is 16,300 lb., and the maximum average unit shear is

$$\frac{16{,}300}{2\frac{1}{2} \times 12} = 540 \text{ lb. per sq. in.}$$

The uprights will be the same as at the expansion end. The unsupported length is 7 inches, which is not sufficient to require an investigation as a short column.

24. **General Drawings.**—The general drawings are shown in Fig. 16.



GENERAL PLAN OF
70'-0" x 16'-0" SPAN
LOW TRUSS HIGHWAY BRIDGE
Designed under
Ketchum's Specifications
Class $D_2$

# CHAPTER XIV.

## DESIGN OF HIGH TRUSS STEEL HIGHWAY BRIDGES.

**Introduction.**—The different types of trusses in use for high truss bridges are described in Chapter VIII. Through truss bridges with spans of from 80 to 160 feet are built with parallel chords, and with either pin-connected or riveted joints. For spans of 160 to 200 feet, bridges are

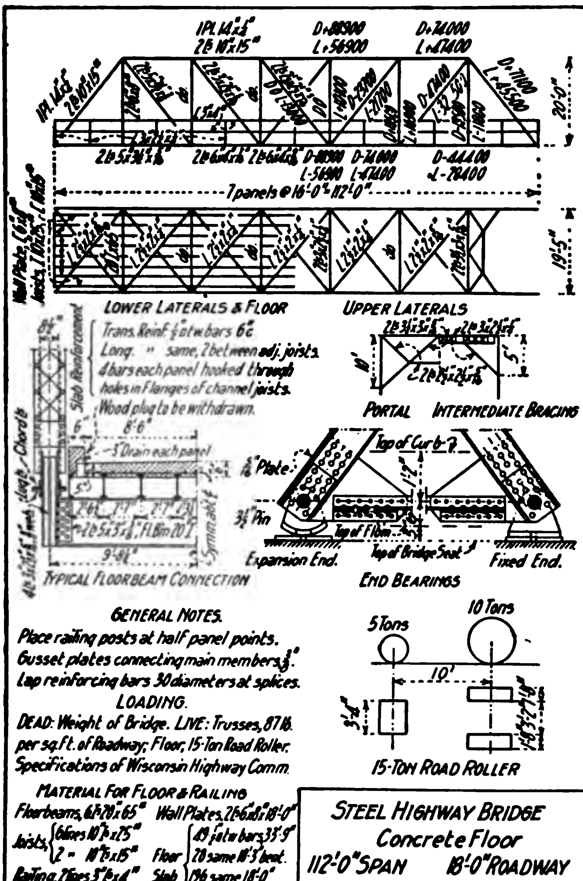



FIG. 3. HIGH TRUSS STEEL HIGHWAY BRIDGE. WISCONSIN HIGHWAY COMMISSION.

usually built of the Pratt type with inclined upper chord (camel-back) trusses. For spans of 200 feet and over, bridges are usually built with subdivided panels. The Petit type of truss has

## TABLE I.

### SUMMARY OF WEIGHT OF METAL.

### 111′ 6″ × 18′ 0″ RIVETED HIGHWAY BRIDGE.

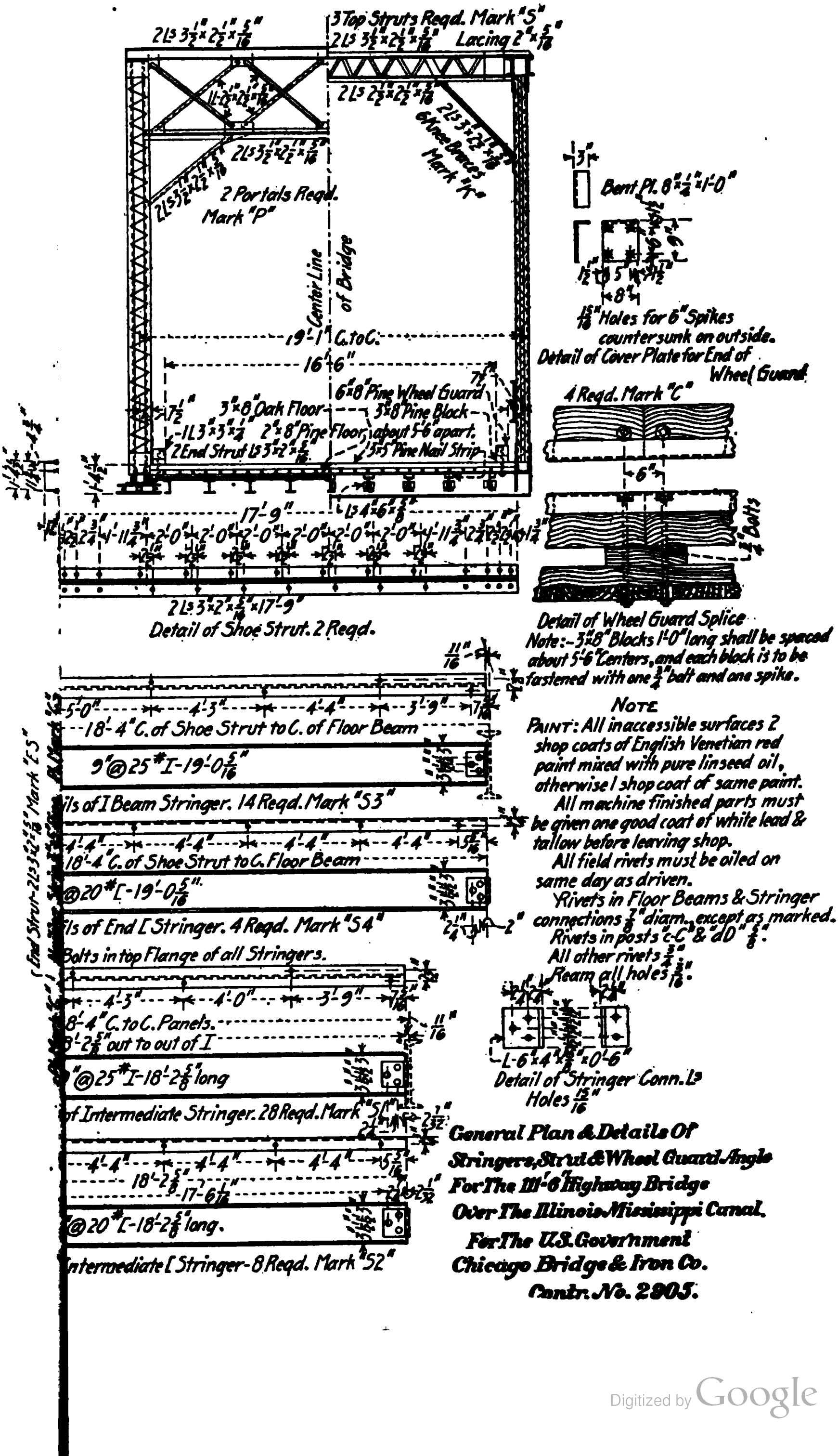
| Ref. No. | Member. | Weights. Main Members. | Details. | Total. | Details Per Cent of Main Members. |
|---|---|---|---|---|---|
| 1 | End-posts. | 5,592 | 3,892 | 9,484 | 67.0 |
| 2 | Top Chords. | 5,900 | 3,942 | 9,842 | 67.0 |
| 3 | Lower Chords. | 5,232 | 442 | 5,674 | 8.5 |
| 4 | Intermediate Posts. | 2,436 | 2,835 | 5,277 | 116.0 |
| 5 | Main Ties. | 3,184 | 474 | 3,658 | 15.0 |
| 6 | Hip Verticals. | 856 | 163 | 1,019 | 19.0 |
| 7 | Counters. | 1,156 | 109 | 1,265 | 9.0 |
| 8 | Floorbeams. | 8,350 | 2,230 | 10,580 | 27.0 |
| 12 | Struts. | 1,486 | 544 | 2,030 | 36.0 |
| 13 | Top Laterals. | 531 | 35 | 566 | 7.0 |
| 14 | Bottom Laterals. | 843 | 182 | 1,025 | 21.0 |
| 15 | Portals. | 1,732 | 620 | 2,352 | 36.0 |
| 16 | Pins and Nuts. | | 86 | 86 | |
| 17 | Pedestals. | | 1,949 | 1,949 | |
| | | 37,298 | 17,503 | 54,801 | 46.9 |
| Total Weight of Metal in Bridge, exclusive of 9, 10, 11, 18 and 19 = 54,801 lb. | | | | | |
| 9 | Joists. | 23,852 | 2,200 | 26,052 | 9.0 |
| 10 | Hub Guard. | 2,392 | .267 | 2,659 | 11.0 |
| 11 | End Struts. | 469 | 167 | 636 | 36.0 |
| 18 | Bolts for Lumber. | | 365 | 365 | |
| 19 | Spikes for Lumber. | | 389 | 389 | |
| | | 26,713 | 3,388 | 30,101 | 13.0 |
| Total Metal in Bridge. | | 64,011 | 20,891 | 84,902 | 33.0 |

## TABLE II.

### DATA ON STANDARD STEEL HIGH TRUSS HIGHWAY BRIDGES.

### WISCONSIN HIGHWAY COMMISSION.

| Span, Ft. | Height of Truss, Ft. | Number of Panels. | Weight of Structural Steel in Trusses, Floorbeams and Lateral Bracing. 16 Ft. Roadway, Lb. | 18 Ft. Roadway, Lb. |
|---|---|---|---|---|
| 90 | 18 | 6 | 34,200 | 37,800 |
| 96 | 18 | 6 | 36,500 | 39,820 |
| 100 | 20 | 6 | 38,900 | 43,370. |
| 105 | 20 | 7 | 42,500 | 46,000 |
| 112 | 20 | 7 | 52,800 | 56,300 |
| 120 | 20 | 8 | 54,900 | 59,700 |
| 128 | 22 | 8 | 58,300 | 61,040 |
| 140 | {20<br>27 | 8 | 65,800 | 71,870 |
| 150 | {20<br>28 | 8 | 70,500 | 76,680 |
| Reinforcing per lineal foot | | | 40 lb. | 43 lb. |
| Railing per lineal foot | | | 19 lb. | 19 lb. |
| Concrete per lineal foot | | | 0.296 cu. yd. | 0.347 cu. yd. |


3 Top Struts Reqd. Mark "S"
Lacing 2" x 5/16"
2 Portals Reqd.
Mark "P"
6 Knee Braces
Mark "K"
Center Line of Bridge
19'-1" C. to C.
16'-6"
6"x8" Pine Wheel Guard
3"x8" Oak Floor
3"x8" Pine Block
2"x8" Pine Floor, about 5'-6" apart.
3"x5" Pine Nail Strip
2 End Strut Ls 3"x2"x5/16"
17'-9"
Detail of Shoe Strut. 2 Reqd.
Bent Pl. 8"x1/4"x1'-0"
15/16" Holes for 6" Spikes countersunk on outside.
Detail of Cover Plate for End of Wheel Guard
4 Reqd. Mark "C"
Detail of Wheel Guard Splice
Note:- 3"x8" Blocks 1'-0" long shall be spaced about 5'-6" Centers, and each block is to be fastened with one 3/4" bolt and one spike.
18'-4" C. of Shoe Strut to C. of Floor Beam
9"@25# I-19'-0 5/16"
Details of I Beam Stringer. 14 Reqd. Mark "S3"
18'-4" C. of Shoe Strut to C. Floor Beam
@20# [-19'-0 5/16"
Details of End [ Stringer. 4 Reqd. Mark "S4"
Bolts in top Flange of all Stringers.
18'-4" C. to C. Panels.
18'-2 5/8" out to out of I
"@25# I-18'-2 5/8" long
of Intermediate Stringer. 28 Reqd. Mark "S1"
@20# [-18'-2 5/8" long.
Intermediate [ Stringer - 8 Reqd. Mark "S2"
NOTE
PAINT: All inaccessible surfaces 2 shop coats of English Venetian red paint mixed with pure linseed oil, otherwise 1 shop coat of same paint.
All machine finished parts must be given one good coat of white lead & tallow before leaving shop.
All field rivets must be oiled on same day as driven.
Rivets in Floor Beams & Stringer connections 7/8" diam., except as marked.
Rivets in posts "C" & "D" 5/8".
All other rivets 3/4".
Ream all holes 1/16".
L-6"x4"x1/2"x0'-6"
Detail of Stringer Conn. Ls
Holes 15/16"
General Plan & Details Of
Stringers, Strut & Wheel Guard Angle
For The 111'-6" Highway Bridge
Over The Illinois Mississippi Canal.
For The U.S. Government
Chicago Bridge & Iron Co.
Contr. No. 2905.

been commonly used for long span bridges, but the K-type of truss has the advantage of smaller secondary stresses and simplicity of design and ease of erection, and is rapidly coming into use. High truss pin-connected bridges should never be built with less than five panels.

The Iowa Highway Commission uses high trusses for spans of 100 ft. and over, riveted with parallel chords to 140 ft.

The Illinois Highway Commission uses high trusses for spans of 90 ft. and over, riveted with parallel chords to 160 ft. For spans longer than 160 ft. trusses may be riveted or pin-connected with parallel or inclined chords.

The Massachusetts Public Service Commission uses high trusses for spans of 100 ft. and over, riveted to 125 ft., and riveted or pin-connected for spans over 125 ft.

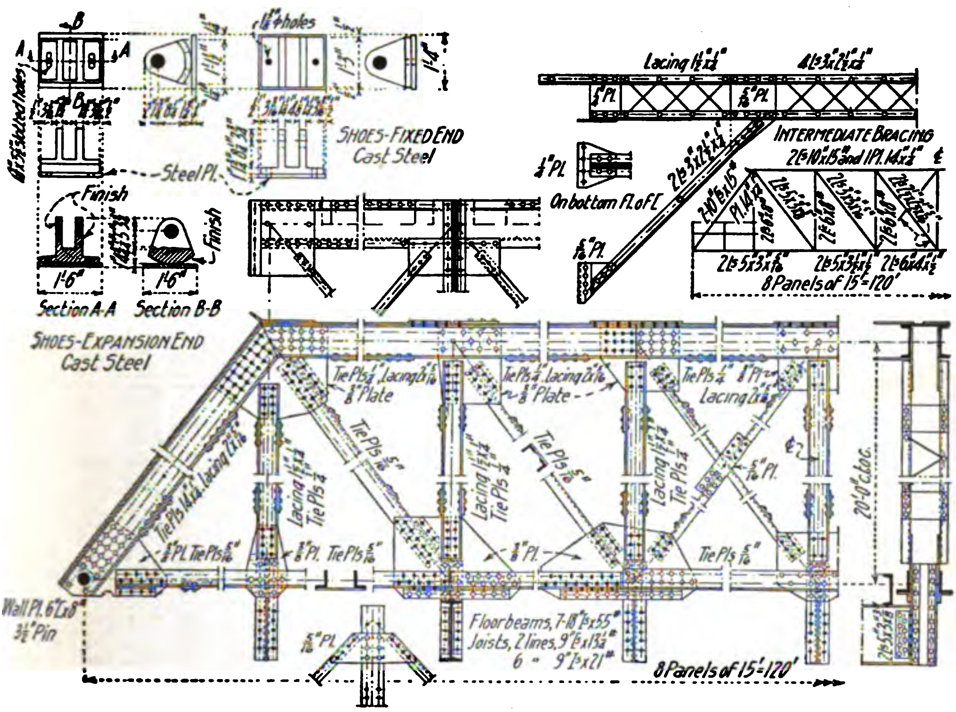


FIG. 4. DETAIL PLANS OF THROUGH TRUSS SPAN. WISCONSIN HIGHWAY COMMISSION.

The Wisconsin Highway Commission uses high trusses of the Pratt type with riveted connections for spans of 80 to 150 ft.

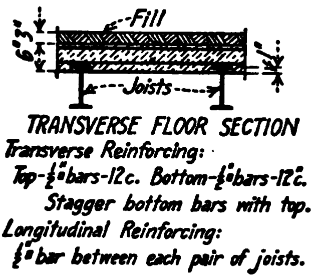
The American Bridge Company has prepared standards of high trusses of the Warren type with spans of 104 to 204 ft. with riveted connections. The bridges have parallel chords up to 150 ft. and inclined chords for longer spans.

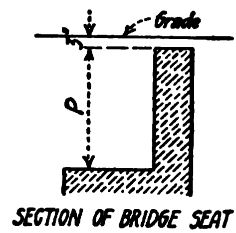
**RIVETED HIGHWAY BRIDGES.**—The detail shop plans of a 111-ft. 6-in. span riveted Pratt truss highway bridge, as built for the U. S. Government by the Chicago Bridge and Iron Co., Chicago, Ill., are shown in Fig. 1 and Fig. 2. The top chords, the end-posts and the intermediate posts are made of two channels laced on both sides, while the bottom chords, hip verticals and diagonal ties are made of two angles fastened together with tie plates. The floor-beams are 18" @ 55 lb. I beams and are riveted below the lower chords. The joists are carried

by connection angles riveted to the webs of the floorbeams. The portals and the sway struts are made of angles. The top and bottom laterals are made of adjustable rods. The expansion end of the bridge is carried on two nests of expansion rollers, each nest being composed of four 3½ in. rollers. The floor covering is composed of a bottom layer of 2 in. × 8 in. pine plank laid transversely and spiked to 3 in. × 5 in. spiking strips that are bolted to the tops of the joists, and a top layer of 3 in. × 8 in. oak, laid diagonally. The 6 in. × 8 in. pine felloe (wheel) guard has its edge protected by an angle 3″ × 3″ × ¼″. The detailed estimate of the weight of this bridge is given in Table I. The per cent of details in this bridge is quite high, due to the fact that the end-posts and the top chords are made of two channels, laced.

## TABLE III.

Standard Steel Through Truss Spans—General Data and Estimated Quantities, Iowa Highway Commission.




| Span and Roadway. | Height of Truss. | Number of Panels. | Center to Center of Trusses. | Height of Parapet, P. | Structural Steel. Trusses, Bracing, Floorbeams, etc. | Structural Steel. Joists. | Structural Steel. Total. | Floor. Reinforcing. | Floor. Concrete. |
|---|---|---|---|---|---|---|---|---|---|
| ft. | ft. | | ft. in. | ft. in. | lb. | lb. | lb. | lb. | cu. yd. |
| 90 × 16 | 20 | 5 | 17 3 | 3 0$\frac{7}{16}$ | 39,200 | 14,500 | 53,700 | 2,960 | 27.3 |
| 90 × 18 | 20 | 5 | 19 3 | 3 4½ | 43,900 | 16,500 | 60,400 | 3,370 | 30.8 |
| 100 × 16 | 20 | 6 | 17 3 | 3 1 | 49,300 | 16,200 | 65,500 | 3,270 | 30.1 |
| 100 × 18 | 20 | 6 | 19 3 | 3 5$\frac{1}{16}$ | 54,300 | 18,400 | 72,700 | 3,710 | 33.9 |
| 110 × 16 | 20 | 6 | 17 3 | 3 0[illegible] | 53,400 | 17,700 | 71,100 | 3,600 | 33.1 |
| 110 × 18 | 20 | 6 | 19 3 | 3 4[illegible] | 59,000 | 20,100 | 79,100 | 4,070 | 37.2 |
| 120 × 16 | 21 | 7 | 17 4 | 3 1$\frac{1}{16}$ | 64,300 | 19,400 | 83,700 | 3,930 | 36.1 |
| 120 × 18 | 21 | 7 | 19 4 | 3 5[illegible] | 70,600 | 22,000 | 92,600 | 4,460 | 40.8 |
| 130 × 16 | 21 | 7 | 17 4 | 3 1[illegible] | 70,400 | 24,400 | 94,800 | 4,250 | 39.1 |
| 130 × 18 | 21 | 7 | 19 4 | 3 5[illegible] | 77,600 | 27,800 | 105,400 | 4,820 | 44.1 |
| 140 × 16 | 21 | 8 | 17 4 | 3 0[illegible] | 81,300 | 22,600 | 103,900 | 4,570 | 42.0 |
| 140 × 18 | 21 | 8 | 19 4 | 3 4[illegible] | 89,500 | 25,700 | 115,200 | 5,200 | 47.5 |
| 150 × 16 | 28 | 8 | 17 4 | 3 1½ | 87,200 | 28,200 | 115,400 | 4,900 | 45.0 |
| 150 × 18 | 28 | 8 | 19 4 | 3 5[illegible] | 96,200 | 32,100 | 128,300 | 5,570 | 50.9 |

General design plans and data for a high truss steel highway bridge with a span of 112 ft. and an 18 ft. roadway, as designed by the Wisconsin Highway Commission are given in Fig. 3. Detail drawings of a 120 ft. span high truss bridge are given in Fig. 4. Standard plans have been prepared for spans of from 90 to 150 ft., with 16 ft. and 18 ft. roadway. All spans have one end carried on rockers as shown. Data on these standard bridges are given in Table II. These designs have been worked out very economically by Mr. M. W. Torkelson, bridge engineer, and represent about the extreme economy in design that will conform to good practice.

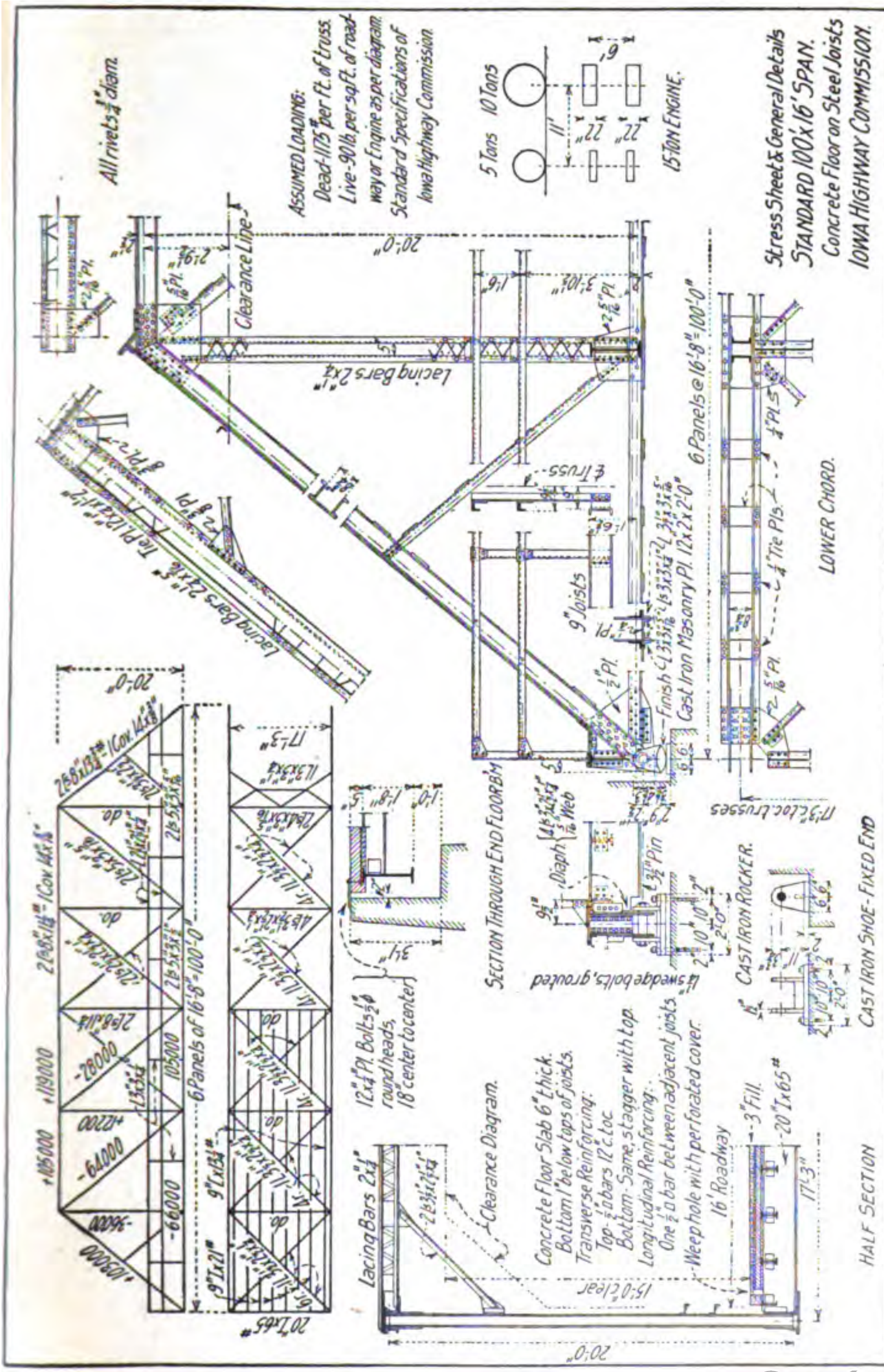



FIG. 5. STANDARD HIGH TRUSS STEEL HIGHWAY BRIDGE. IOWA HIGHWAY COMMISSION.

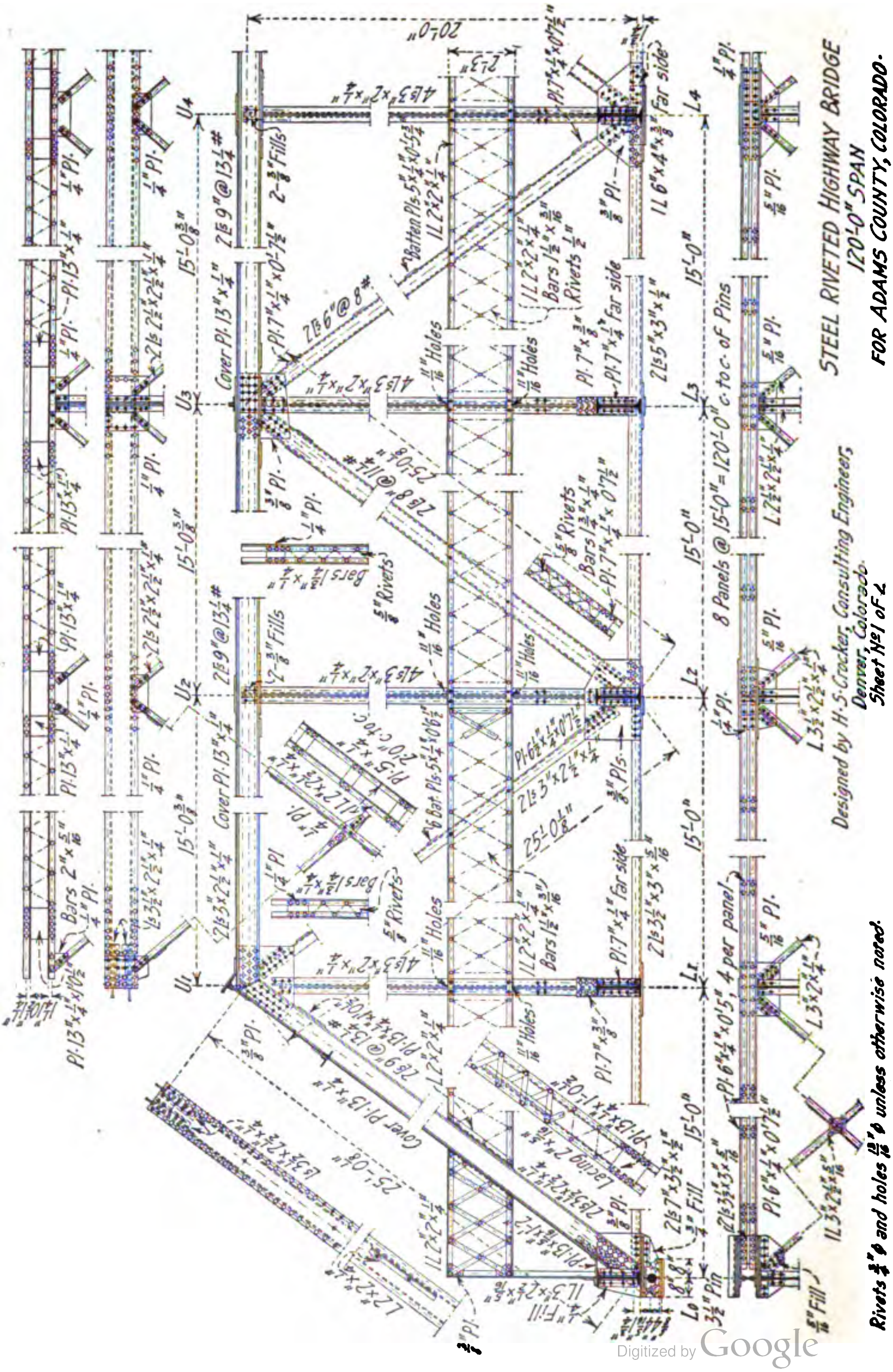



FIG. 6. DETAILS OF A THROUGH RIVETED HIGHWAY BRIDGE.

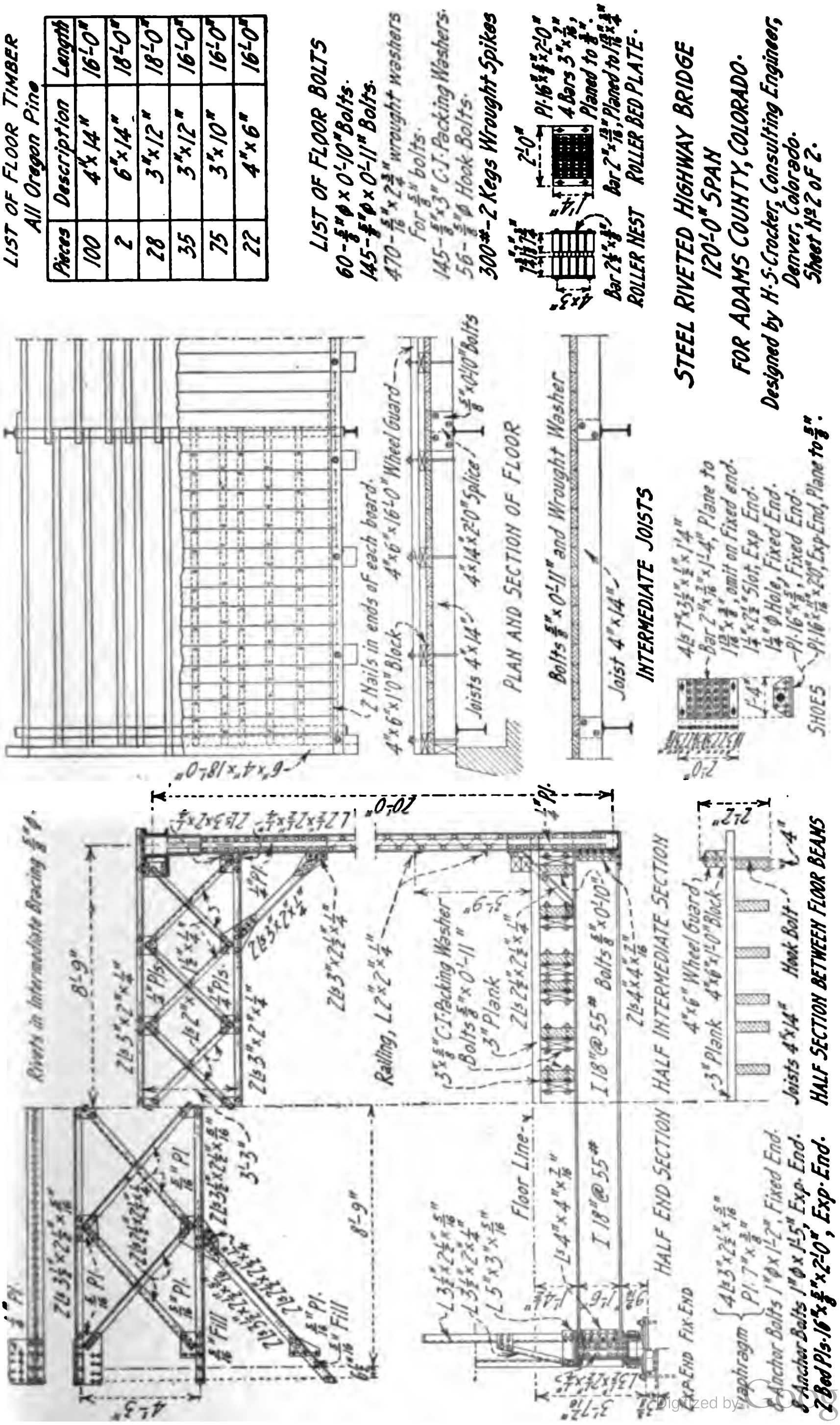



FIG. 7. DETAILS OF A THROUGH RIVETED HIGHWAY BRIDGE.

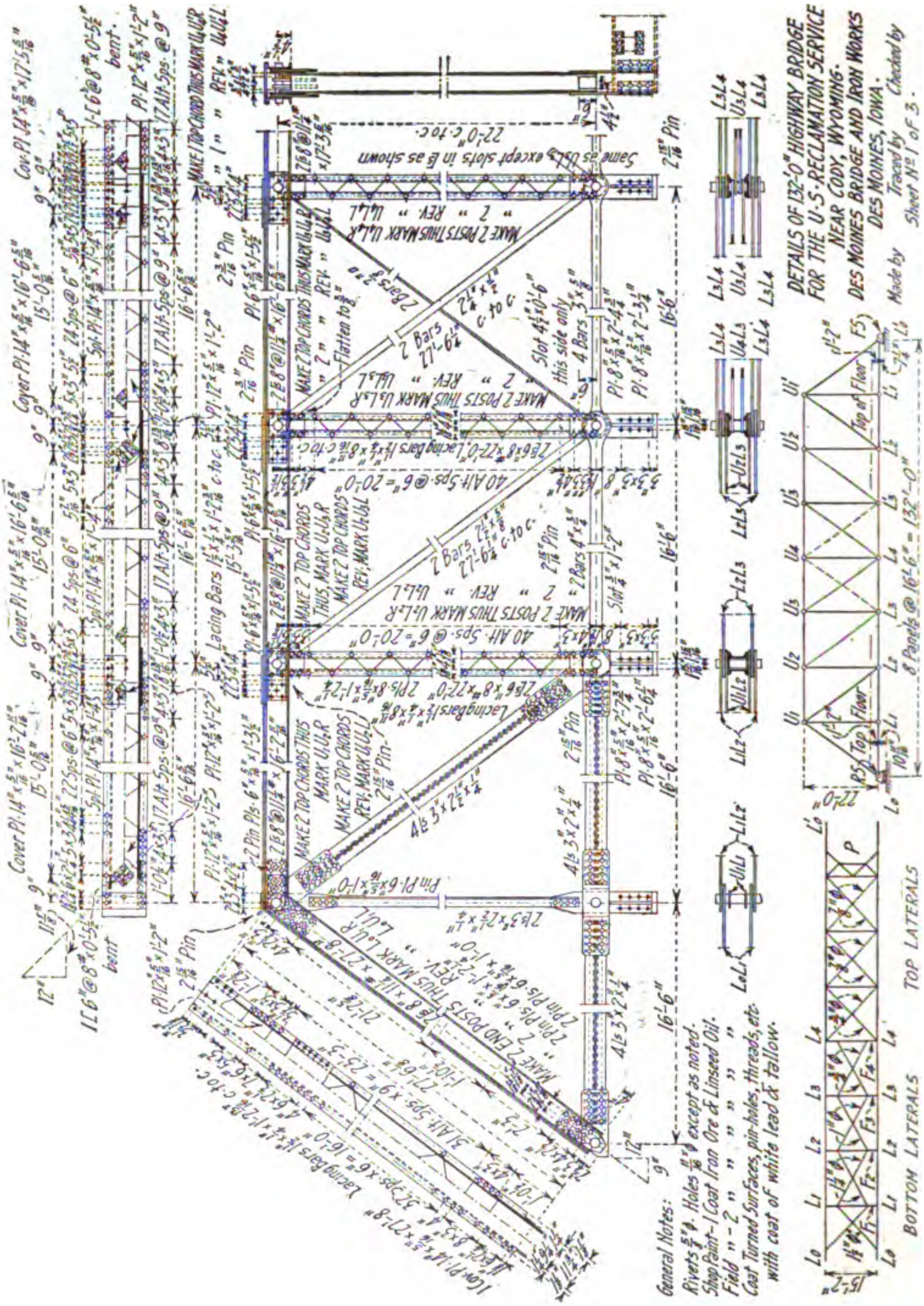



FIG. 8. DETAILS OF A PIN-CONNECTED HIGHWAY BRIDGE.

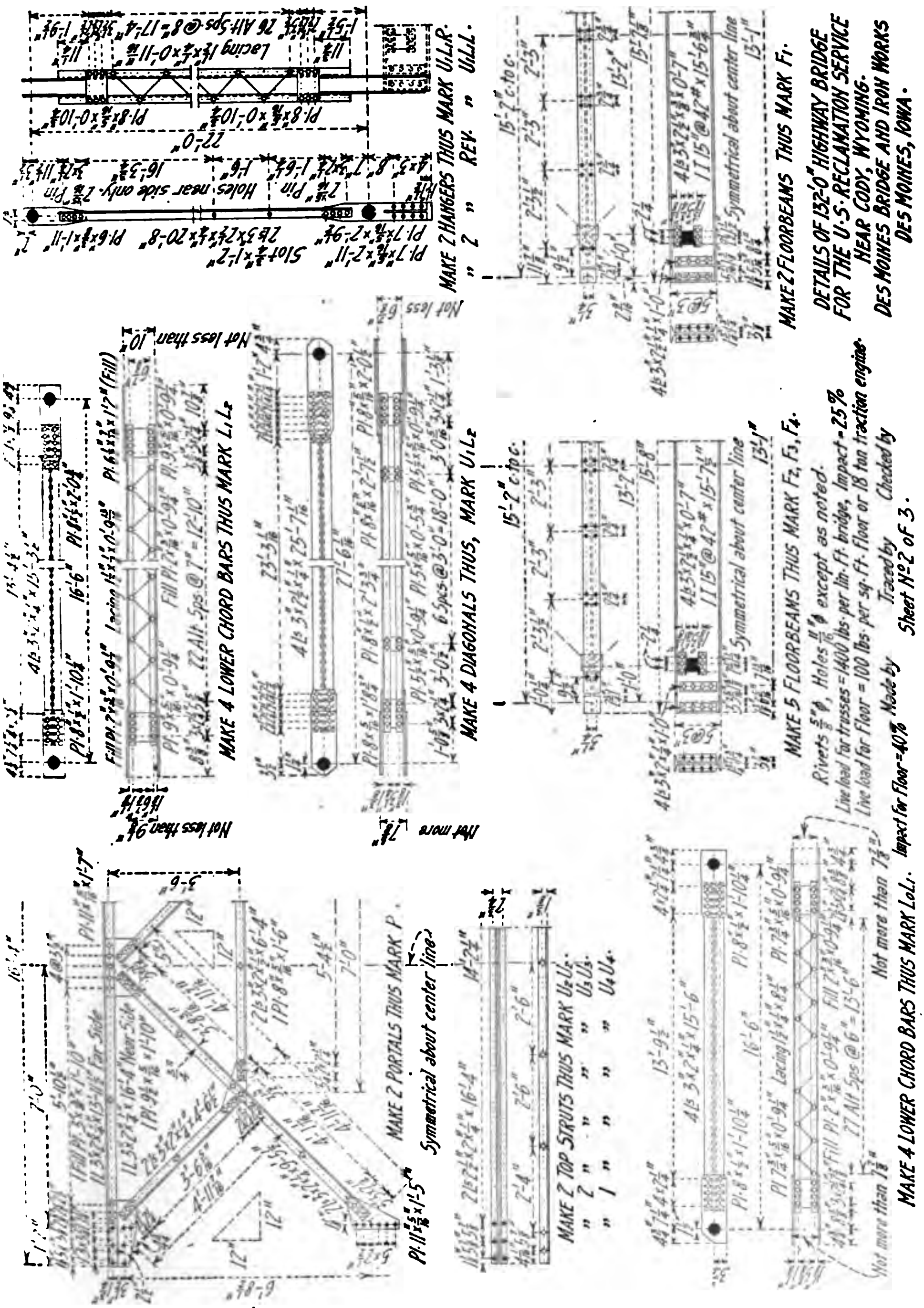



FIG. 9. DETAILS OF A PIN-CONNECTED HIGHWAY BRIDGE.

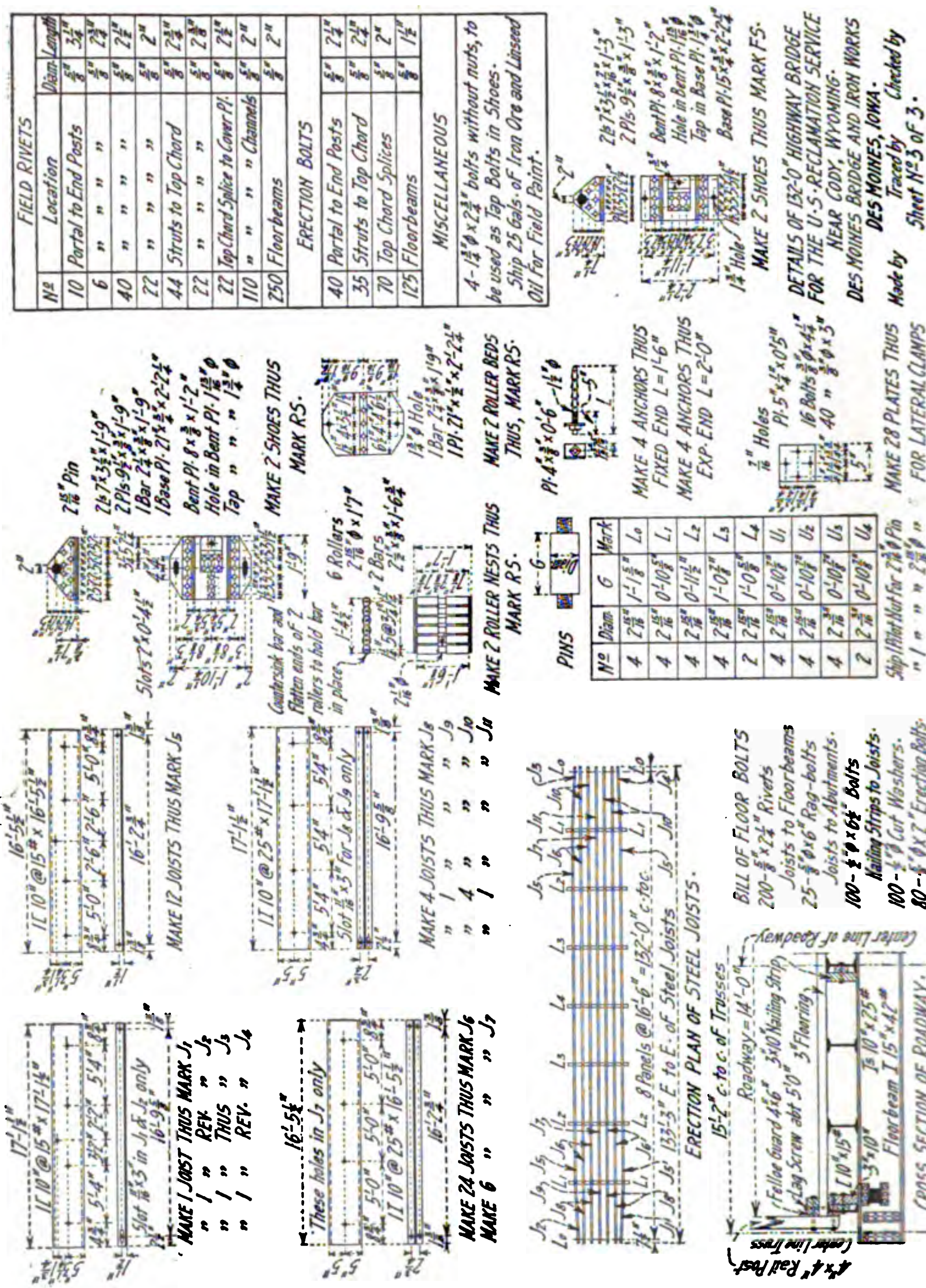



FIG. 10. DETAILS OF A PIN-CONNECTED HIGHWAY BRIDGE.

Details of a high truss steel highway bridge as designed by the Iowa Highway Commission are given in Fig. 5. Standard plans have been prepared for spans of 90 ft. to 150 ft., varying by 10 ft. intervals, and with roadways of 16 ft. and 18 ft. All spans have one end carried on rockers as shown. Data on these standard bridges are given in Table III. The designs are well worked out and represent good practice, except that the collision strut in the first panel should be omitted.

Details of a 120 ft. span riveted truss steel highway bridge designed by Mr. H. S. Crocker, consulting engineer, Denver, Colorado, are shown in Fig. 6 and Fig. 7. This bridge has a timber floor carried on timber joists. The expansion end of the bridge is carried on 3 in. steel rollers. This bridge was designed for country roads and represents good practice.

**PIN-CONNECTED HIGHWAY BRIDGES.**—Shop details of a 132-ft. span pin-connected steel highway bridge designed for the Reclamation Service are given in Fig. 8, Fig. 9, and Fig. 10. The 3-in. timber floor is carried on steel joists. The expansion end of the bridge is carried on steel rollers. The details of this bridge are well worked out, and the plans are very complete. The estimated weight of this span not including steel joists and fence was 44,670 lb.

The general drawings and stress sheet for the 224 ft. by 18 ft. span steel truss highway bridge built over the Big Vermillion River in La Salle County, Illinois is shown in Fig. 11. This crossing consists of one 224 ft. steel truss bridge and four 45 ft. reinforced concrete through girder spans, on reinforced concrete piers and abutments. A view of the complete bridge is shown in Fig. 4, Chapter XVII. This bridge was designed in 1914 under the specifications for steel highway bridges prepared by the Illinois Highway Commission. The allowable tensile stress in eye-bars was 18,000 lb. per sq. in. The 4-in. reinforced concrete slab floor was covered with a 4-in. concrete wearing surface.

**Economic Depth and Panel Length of Trusses.**—The economic depth and panel length of trusses is not capable of mathematical calculation. The minimum depth is determined by the required clear head room, which varies from 12½ to 15 ft. Short panel lengths give heavy trusses and light floor systems; while long panels give light trusses and heavy floor systems. For ordinary conditions it is not economical to use panel lengths less than 15 ft. for short spans nor more than 25 ft. for long spans. The minimum depth for through spans is about 16 feet where the floorbeams are placed below the lower chords. To make a stiff structure, the depth should be sufficient to permit the placing of the floorbeams above the lower chords and to permit of efficient portal and sway bracing. Experience has shown that the most economical conditions occur when the angle $\theta$, the tangent of which is the panel length divided by the depth, is about 40 degrees. The top chord points of bridges with inclined chords should be approximately on a parabola passing through the pin at the hip. For a discussion of economic span of bridges, see Chapter XV.

**Depth and Panel Length of High Trusses.**—The depths and number of panels in Iowa Highway Commission high truss riveted bridges are as follows: Pratt, riveted trusses, 90-ft. span, 5 panels, 20 ft. deep; 100-ft. and 110-ft. spans, 6 panels, 20 ft. deep; 120-ft. span, 7 panels, 20 ft. deep; 140-ft. span, 8 panels, 21 ft. deep. The depths and number of panels in Wisconsin Highway Commission high truss riveted bridges are as follows: 90 ft. and 96 ft. span, 6 panels, 18 ft. deep; 100-ft. span, 6 panels, 20 ft. deep; 105-ft. span, 7 panels, 20 ft. deep; 120-ft. span, 8 panels, 20 ft. deep; 128-ft. span, 8 panels, 21 ft. deep; 140 -ft.span, 8 panels, 20 ft. deep, at hip and 27 ft. deep at center; 150-ft. span, 8 panels, 20 ft. deep at hip and 28 ft. deep at center.

The depths and number of panels in American Bridge Company's high truss bridges are as follows: Riveted and pin-connected trusses with parallel chords, 80-ft. to 90-ft. span, 5 panels, depth equal to panel length; 90-ft. to 120-ft. span, 6 panels, depth equal to panel length; 120-ft. span to 140-ft. span, 7 panels, depth equal to panel length, 120-ft. to 168-ft. span, 8 panels, ratio of depth to panel length 1.1. For bridges with inclined chords with spans of 162 ft. to 180 ft., 9 panels, and ratios of depth to panel length of 1.0 ,1.16, 1.25 and 1.29; 190-ft. to 220-ft. span, 9 panels and ratios of depth to panel length of 1.0, 1.24, 1.28 and 1.43. For Petit trusses, 240-ft. to 276-ft. span, 12 panels, and ratios of depths to panel length of 1.0, 1.4, 1.6 and 1.7; 294-ft. to 322-ft. span, 14 panels, and ratios of depth to panel length of 1.0, 1.36, 1.60, 1.8 and 2.0.

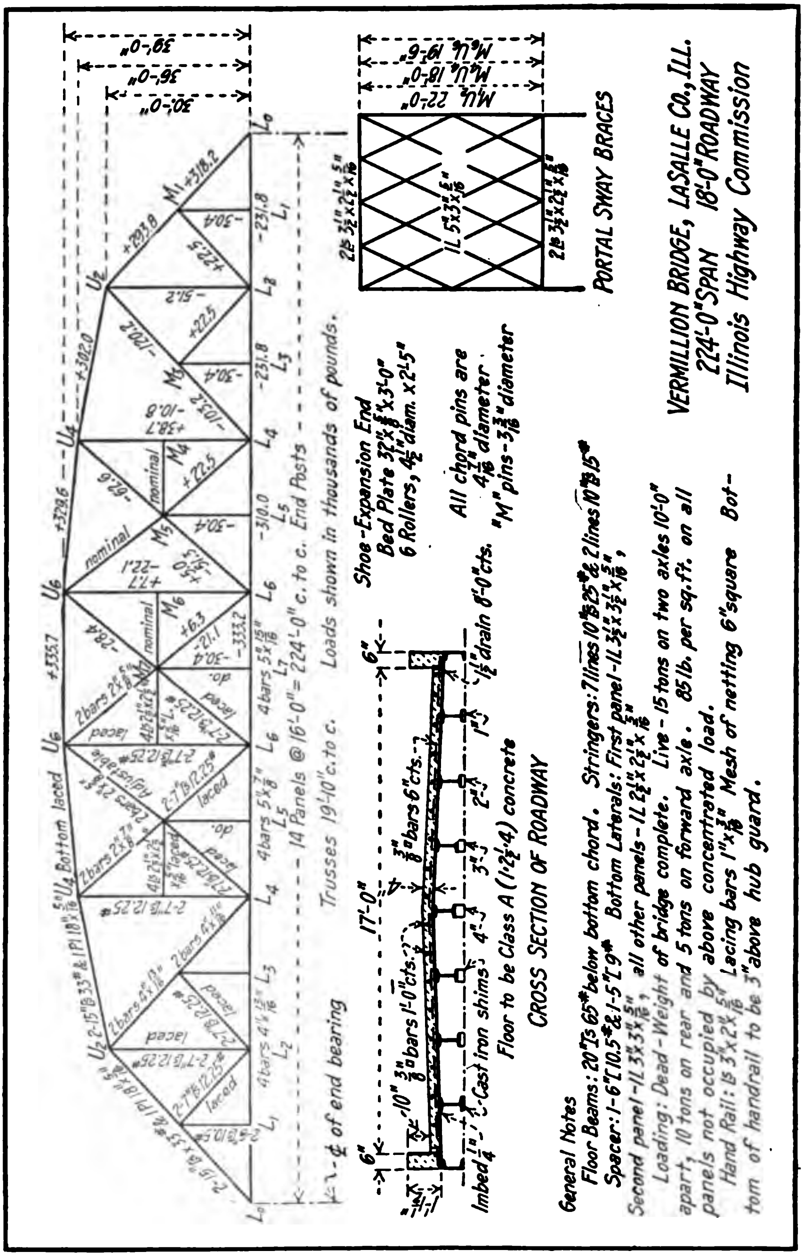

VERMILLION BRIDGE, LASALLE CO., ILL.
224'-0" SPAN 18'-0" ROADWAY
Illinois Highway Commission
PORTAL SWAY BRACES
CROSS SECTION OF ROADWAY
Shoe-Expansion End
Bed Plate 32"×7/8"×3'-0"
6 Rollers, 4½" diam. × 2'-5"
All chord pins are 4 7/16" diameter
"M" pins-3 3/16" diameter
14 Panels @ 16'-0" = 224'-0" c. to c. End Posts
Trusses 19'-10" c. to c. Loads shown in thousands of pounds.
Floor to be Class A (1:2½:4) concrete
General Notes
Floor Beams: 20" I's 65# below bottom chord. Stringers: 7 lines 10" I's 25# & 2 lines 10" [ 15#
Spacer: 1-6" [ 10.5# & 1-5" [ 9# Bottom Laterals: First panel-1 L 3½"×3½"×5/16", Second panel-1 L 3"×3"×5/16", all other panels-1 L 2½"×2½"×5/16",
Loading: Dead-Weight of bridge complete. Live-15 tons on two axles 10'-0" apart, 10 tons on rear and 5 tons on forward axle. 85 lb. per sq. ft. on all panels not occupied by above concentrated load.
Hand Rail: L 3"×2½"×5/16" Lacing bars 1"×3/16" Mesh of netting 6" square Bottom of handrail to be 3" above hub guard.

FIG. 11.

THE DESIGN OF A 112 FT. SPAN HIGH RIVETED PRATT TRUSS BRIDGE.

1. **General Description of Bridge.**—This is to be a through bridge having riveted Pratt trusses with parallel chords. The floor is to be composed of a reinforced concrete slab, with additional wearing surface, placed on I-beam joists.

2. **LOADS. Dead Load.**—The dead load consists of the weight of the reinforced concrete floor slab at 150 lb. per cu. ft., the joists, floorbeams, trusses and lateral bracing.

**Live Load.**—This bridge will be designed for Class $D_1$ loading which provides for a 20-ton concentrated load or a uniform load of 100 lb. per sq. ft. of roadway for the floor and its supports. The live load for the trusses is given in Table I of the specifications in Appendix I as 78 lb. per sq. ft. of roadway.

**Impact.**—The specifications provide for an allowance for impact of 30 per cent of the live load for the floor and its supports, and of

$$\frac{100}{L + 300} = \frac{100}{112 + 300} = 24.3 \text{ per cent}$$

for the truss members.

**Wind Load.**—The specifications require that the lower lateral bracing be designed for a moving wind load of 300 lb. per foot of bridge, and the upper lateral bracing be designed for a moving wind load of 150 lb. per foot of bridge.

3. **Dimensions.**—Span, 112′ 0″ c. to c. of end bearings; panel length, 16′ 0″; width of roadway 16′ 0″; spacing of trusses, 17′ 3″ c. to c.

4. **Depth of Trusses.**—The trusses must have a depth sufficient to provide head room of 15 ft. for a width on the center line of bridge of 8 ft. The bottom of the floorbeam will be placed even with the bottom of the lower chord angles or about 2 in. below the center line of the chord. The floorbeam will probably have a depth of 20 in. and the slab a depth of 6 in. See Table I, Chapter X. The top of the joist will be even with the top of the floorbeam. Using a depth of 20 ft. c. to c. of chords the various parts will occupy the following depths.

| | |
|---|---|
| Headroom | 15′ 0″. |
| Floorbeam | 1′ 6″. |
| Slab | 0′ 6″. |
| Portal | 3′ 0″. |
| Total | 20′ 0″. |

5. **DESIGN OF FLOOR SYSTEM.**—The methods used in the design of the floor system of this bridge are given in Chapter X, and will not be repeated.

The slab will be found in Table I, and the joists and floorbeams will be found in Fig. 8, Chapter X.

The following floor system will be used:

Slab; total thickness 6 in., depth to center of steel 5 in. Reinforcement at right angles to joist; ½ in. square rods 5½ in. c. to c. Reinforcement parallel to joists; two ½ in. square rods between adjacent joists.

Joists, 10 in. I's @ 25 lb. spaced about 2 ft. 2 in. c. to c. Intermediate floorbeams, 20 in. I's @ 80 lb. End floorbeams, 20 in. I's @ 65 lb.

The estimated weight of the floor system is 25,500 lb. per panel.

In order to calculate the live load floorbeam reactions it is necessary to calculate, first, the maximum load that can come on a floorbeam and second, the maximum reaction that can occur due to this load. From (*a*), Fig. 13, the greatest live load that can come on the floorbeam will be

$14 + 4 \times 6/16 = 15.5$ tons. From (*b*), Fig. 13, it will be seen that the maximum reaction will occur with the wheels as shown. (The truck is assumed as 10 feet wide.)

$$R_1 = \frac{11.63 \times 15.5}{17.25} \times 2{,}000 = 20{,}900 \text{ lb.}$$

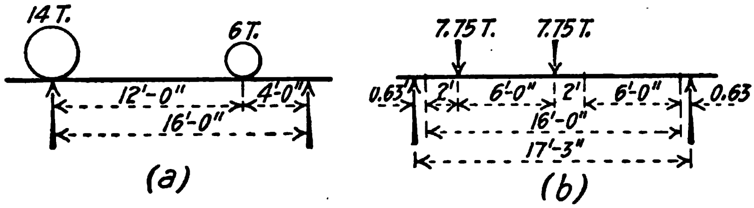



FIG. 13.

Allowing 30 per cent for impact the reaction will be

$$1.30 \times 20{,}900 = 27{,}200 \text{ lb.}$$

| | Intermediate. | End. |
|---|---|---|
| D. L. floorbeam reaction | 12,750 lb. | 6,375 lb. |
| L. L. and Impact floorbeam reaction | 27,200 " | 27,200 " |
| Total floorbeam reaction | 39,950 lb. | 33,575 lb. |

6. **Joint Loads.**—*Dead Load.*

| | |
|---|---|
| Trusses (including Bracing, Portals, Laterals and railing) | 55,600 lb. |
| Joists | 18,000 " |
| Floor system (estimated) | 160,400 " |
| Total | 234,000 lb. |

Dead joint load, $234{,}000/14 = 16{,}700$ lb.
Live Load, $78 \times 16 \times 16 \div 2 = 10{,}000$ lb. per joint.
Impact, $10{,}000 \times 0.243 = 2{,}430$ lb. per joint.
*Wind load.*—Upper chord, $150 \times 16 = 2{,}400$ lb. per joint as a moving load.
Lower chord, $300 \times 16 = 4{,}800$ lb. per joint as a moving load.

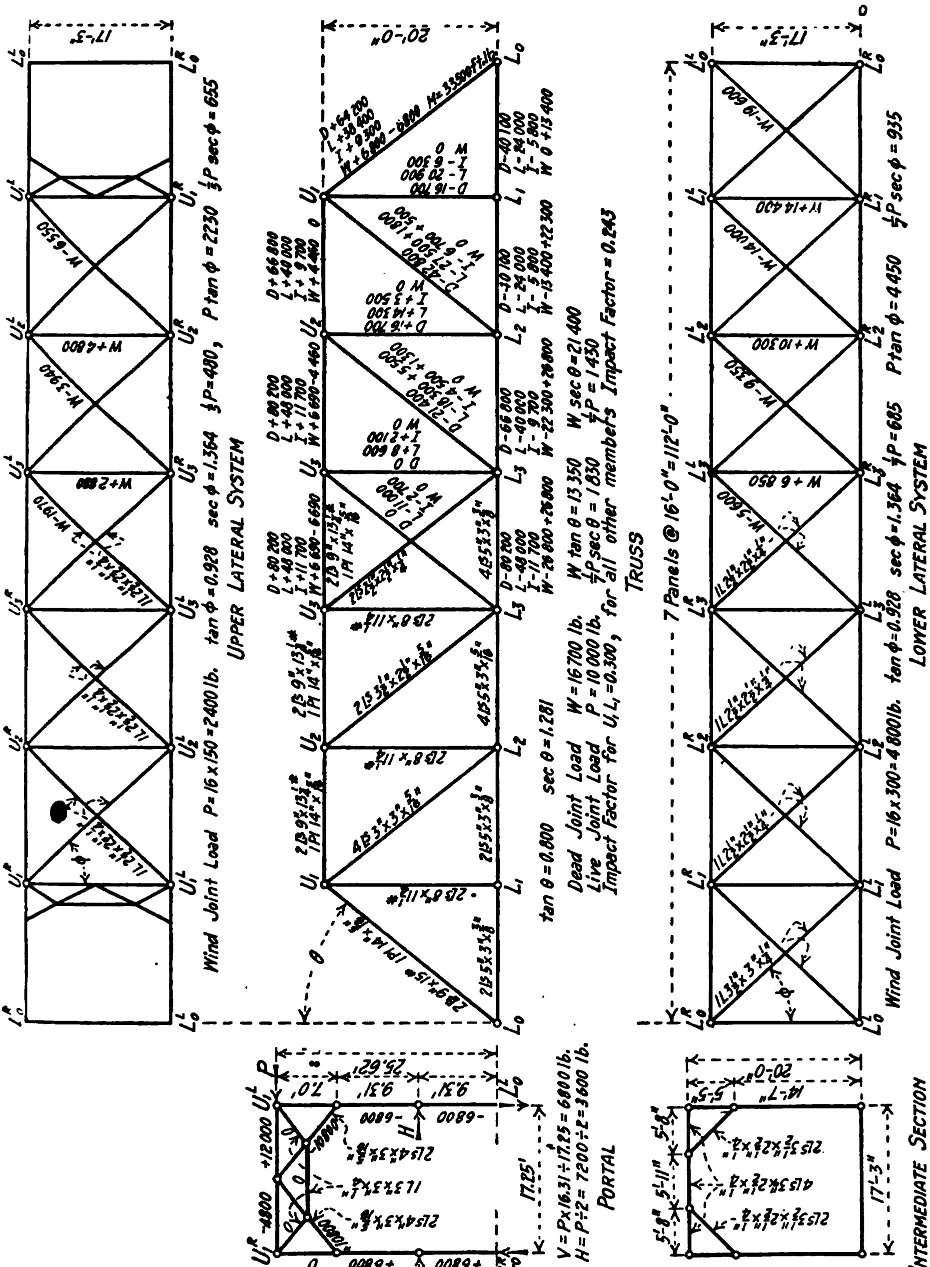
7. **Dead Load Stresses.**—The dead load stresses as calculated by the method of coefficients are shown on the stress sheet in Fig. 14.

8. **Live Load and Impact Stresses.**—The live load stresses and impact stresses are shown on the stress sheet in Fig. 14. The live load stresses were calculated by the method of coefficients and the impact stresses were calculated by multiplying the live load stresses by the impact factor 0.243.

The stress in the hanger $U_1L_1$ is equal to the end shear of the floorbeam as calculated in the design of the floor system, and is 20,900 lb. for live load only, and 27,200 lb. including 30 per cent impact.

9. **Wind Load Stresses.**—In the calculation of the stresses due to wind the entire wind load on the top chord will be carried along the top chord to the portal by the top lateral system, none of it being considered as carried down the posts to the bottom lateral system, the sway bracing at the intermediate posts being made light.

The total wind load is considered as concentrated at the joints of the windward truss for both upper and lower lateral systems. One-half of the load is sometimes considered as acting on the leeward side, especially for the unloaded chord since it is not sheltered by the floor system. This




STRESS SHEET

FIG. 14.

distribution does not affect the stresses other than those in the lateral struts and the effect there is negligible. All of the wind load shear in any panel of the upper or lower lateral systems is considered as being carried by the member which will take it in tension, the compression in the member with opposite inclination being neglected. The effect of the solid floor is not considered, although after the bridge is completed the lower laterals will not be necessary, as far as wind stresses are concerned.

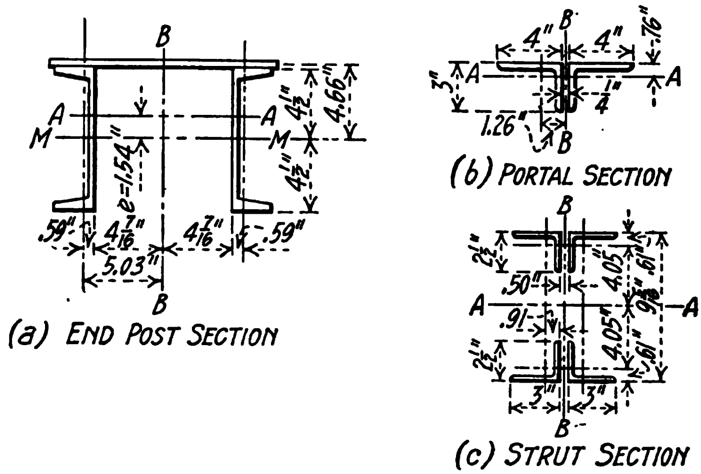



FIG. 15.

The wind load stresses as calculated by the method of coefficients are shown on the stress sheet in Fig. 14.

The portal will be of the type shown in Fig. 14, the horizontal member being placed so as to allow the required headroom of 15 feet. It has been found, par. 4, that the portal may occupy a vertical depth of 3′ 0″, which gives 3.00 sec $\theta$ = 3.85 ft. = 3′ 10″ measured along the end post. Allowing for the distance from the gage line to the lower edge of the member $DE$, a depth of 3′ 6″ will be adopted.

The end-posts will be considered as fixed at the base, § 30, and the point of contraflexure midway between the foot of the main diagonal and $L_0$. The stresses in the portal and end-posts are shown on the stress sheet in Fig. 14.

10. **DESIGN OF TRUSSES AND LATERAL SYSTEMS. Stress Sheet.**—Before proceeding to the design of members the stresses due to the various loadings will be collected on the stress sheet in Fig. 14. As soon as the size of each member is determined it will be recorded on the stress sheet.

In all truss members except $L_0U_1$, the wind stress need not be considered unless it exceeds 25 per cent of the sum of the dead, live and impact stresses. The member $L_0U_1$ is subjected to flexural wind stress which must be considered.

11. **Design of Tension Members.**—Tables IV and V give the design of all tension members in tabular form. The allowable unit tensile stress on the net area is 16,000 lb. per sq. in. when wind does not control, and 1.25 × 16,000 = 20,000 lb. per sq. in. when wind controls. The tension members except $U_1L_1$ will be made of angles. If either leg of an angle is 5 in. or over in width, two holes will be deducted from the section to obtain the net section, otherwise one hole will be deducted.

TRUSS MEMBERS (TENSION).

| Member. | $D+L+I$. | $W$. | $\frac{W}{D+L+I}$. | Allowable Unit Stress. | Required Net Area. | Section. | Actual Net Area. | Rivets on Two Sides at End with ⅜ In. Gusset Plates. Shop. | Field. |
|---|---|---|---|---|---|---|---|---|---|
| $L_0L_1$ | − 69,900 | + 13,400 | 0.19 | 16,000 | 4.37 | 2∠5 × 3½ × ⅜ | 4.78 | 15 | 18 |
| $L_1L_2$ | − 69,900 | { − 13,400 + 22,300 | 0.19 | 16,000 | 4.37 | 2∠5 × 3½ × ⅜ | 4.78 | 15 | 18 |
| $L_2L_3$ | − 116,500 | { − 22,300 + 26,800 | 0.19 | 16,000 | 7.29 | 4∠5 × 3 × 5/16 | 7.40 | 18(1) | 21(1) |
| $L_3L_3'$ | − 139,900 | { − 26,800 + 26,800 | 0.19 | 16,000 | 8.74 | 4∠5 × 3 × ⅜ | 8.80 | 21(1) | 25(1) |
| $U_1L_2$ | − 77,000 | 0 | 0 | 16,000 | 4.81 | 4∠3 × 3 × 5/16 | 5.50 | 16 | 20 |
| $U_2L_3$ | − 44,200 | 0 | 0 | 16,000 | 2.76 | 2∠3½ × 2½ × 5/16 | 3.02 | 6 | 11 |
| $U_3L_3'$ | − 13,700 | 0 | 0 | 16,000 | 0.86 | 2∠2½ × 2½ × ¼(2) | 1.45 | 5(1) | 7(1) |
| $U_1L_1$ | − 43,850 | 0 | 0 | 16,000 | 2.74 | 2 [s 8 @ 11¼ lb. | 5.39 | 16 | 20 |

[1] Bearing will control.

[2] Fastened by one leg.

## TABLE V.

### Lateral Members.

| Member. | Stress. | Allowable Unit Stress. | Req. Net Area. | Section. | Act. Net Area. | Field Rivets at End. |
|---|---|---|---|---|---|---|
| $U_1U_2$ | − 6,550 | 20,000 | 0.33 | 1∠2½ × 2½ × ¼ | 0.73(1) | 3 |
| $U_2U_3$ | − 3,940 | 20,000 | 0.20 | 1∠2½ × 2½ × ¼ | 0.73(1) | 3 |
| $U_3U_3'$ | − 1,970 | 20,000 | 0.10 | 1∠2½ × 2½ × ¼ | 0.73(1) | 3 |
| $L_0L_1$ | − 19,600 | 20,000 | 0.98 | 1∠3½ × 3 × ¼ | 1.0(1) | 4 |
| $L_1L_2$ | − 14,000 | 20,000 | 0.70 | 1∠2½ × 2½ × ¼ | 0.73(1) | 3 |
| $L_2L_3$ | − 9,350 | 20,000 | 0.47 | 1∠2½ × 2½ × ¼ | 0.73(1) | 3 |
| $L_3L_3'$ | − 5,600 | 20,000 | 0.28 | 1×2½ × 2½ × ¼ | 0.73(1) | 3 |

[1] Fastened by one leg, $l/r$ not greater than 150.

## TABLE VI.

### Truss Members (Compression).

| Member. | $D+L+I$. | $W$. | $\frac{W}{D+L+I}$. | $l$ In. | $r$ In. | $l/r$. | Allowable Unit Stress. | Req. Area. | Section. | Actual Area. | Rivets at Ends. Cover Plate. Shop. | Cover Plate. Field. | Channels. Shop. | Channels. Field. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $L_0U_1$ | + 111,900 | 6,800 | .06 | 307 | 3.54 | 86.8 | 9,920 | 11.27 | 2[s 9″ @ 15 lb., 1 Pl. 14″ × 5/16″ | 13.20 | | | 38 | 45 |
| $U_1U_2$ | + 116,500 | + 4,460 | .04 | 192 | 3.57 | 53.8 | 12,230 | 9.53 | 2[s 9″ @ 13¼ lb., 1 Pl. 14″ × 5/16″ | 12.16 | | | 36 | |
| $U_2U_3$ | + 139,900 | + 6,690 | .05 | 192 | 3.57 | 53.8 | 12,230 | 11.44 | 2[s 9″ @ 13¼ lb., 1 Pl. 14″ × 5/16″ | 12.16 | | 12 | | 28 |
| $U_3U_3'$ | + 139,900 | + 6,690 | .05 | 192 | 3.57 | 53.8 | 12,230 | 11.44 | 2[s 9″ @ 13¼ lb., 1 Pl. 14″ × 5/16″ | 12.16 | | 12 | 23 | |
| $U_2L_2$ | + 34,500 | 0 | 0 | 240 | 3.11 | 77.2 | 10,600 | 3.25 | 2[s 8″ @ 11¼ lb. | 6.70 | | | | 18 |
| [illegible] | + [illegible] | 0 | 0 | 240 | 3.11 | 77.2 | 10,600 | 1.01 | 2[s 8″ @ 11¼ lb. | 6.70 | | | | 18 |

Rivets $\frac{3}{4}$ in. in diameter will be used throughout the bridge except in the flanges of channels smaller than 8 in., where $\frac{5}{8}$ in. rivets will be used. In obtaining the net section, holes $\frac{7}{8}$ in. in diameter will be deducted for $\frac{3}{4}$ in. rivets and $\frac{3}{4}$ in. holes for $\frac{5}{8}$ in. rivets. The design of tension members consists mainly of selecting sections to provide the required net area.

In designing all members the form of details used must be kept in mind and the number of shapes and sizes should be made as small as possible. Material which is available without delay should be used.

12. **Design of Compression Members.**—In the design of a compression member it is necessary to select a section, determine its radii of gyration, calculate the allowable unit stress and investigate to see if the section provides the proper gross area. If not a new trial must be made. The greatest value of $l/r$ for compression members must not exceed 125 for main members and 150 for laterals, for this type of bridge. The properties of top chord sections consisting of two channels and a cover plate are given in Table 17, Appendix III.

The explanation of the method followed in determining the number of rivets required at the ends of the tension members, will also apply to compression members.

The design of the compression members of the truss is shown in Table VI.

The centroid of the top chord and end-post sections will be placed on the center line, so no eccentric stresses need be provided for.

The stress in the top chord due to the weight of member should be investigated. The weight of $U_1U_2$, adding 30 per cent for details, is $1.30 \times 12.16 \times 3.4 = 53.8$ lb. per ft. The weight of $U_2U_3$ and $U_3U_3 = 1.30 \times 12.16 \times 3.4 = 53.8$ lb. per ft.

13. **Design of End-post.**—The member $L_0U_1$ carries a direct stress of 111,900 lb. for dead load, live load, and impact. The length of the member for bending about its axis parallel to the cover plate is 25.62 ft. The section will be composed of two 9 in. channels and one 14 in. cover plate. For this section and this axis the radius of gyration will be about 3.55 in., and the allowable unit stress not considering wind, about $16{,}000 - (70 \times 25.62 \times 12/3.55) \doteq 9{,}940$ lb. per sq. in. Approximate radii of gyration of built sections are given in Table 43, Appendix III.

The area required for dead load, live load and impact stress is about $111{,}900 \div 9{,}940 = 11.26$ sq. in. The section composed of 2[s 9 in. @ 15 lb., $8\frac{3}{8}$ in. back to back, and one pl. 14 in. $\times \frac{5}{16}$ in. will be tried. The properties of this section are not given in Table 17, Appendix III, and must be calculated.

Tables of properties of sections may be obtained from the author's "Structural Engineer's Handbook."

| | | |
|---|---|---|
| *Area.* The area of 2 [s | $= 2 \times 4.41 =$ | 8.82 sq. in. |
| The area of a 14 in. $\times \frac{5}{16}$ in. plate | $=$ | 4.38 " " |
| Total area | $=$ | 13.20 sq. in. |

*Centroid.*—The distance from the axis $M$-$M$ to the centroid of the section is found by taking moments about the axis $M$-$M$, and dividing by the area of the section. Moment of channels $= 0$. Moment of plate $= 4.66 \times 4.38 = 20.4$. Total moment $= 20.4$. $e = 20.4/13.20 = 1.54$ in.

*Moment of Inertia. Axis A–A.*—The moment of inertia about the axis $M$-$M$ is first calculated. The value of $I_A$ is obtained by subtracting $A \cdot e^2$ from $I_M$.

| | | |
|---|---|---|
| Channels, $2 \times I = 2 \times 50.9 =$ | | 101.8 in⁴. |
| Plate, area $\times$ square of distance from center to $MM$ | $= 4.38 \times 4.66^2 =$ | 95.0 " |
| Total $I_M$ | $=$ | 196.8 in⁴. |

$$I_A = I_M - A \cdot e^2 = 196.8 - 13.20 \times 1.54^2 = 165.2 \text{ in}^4.$$

*Moment of Inertia Axis B–B.*—

| | | |
|---|---|---|
| Channels, $2(1.95 + 4.41 \times 5.03^2)$ | $=$ | 227.1 in⁴. |
| Plate, $\frac{1}{12}bd^3 = \frac{1}{12} \times \frac{5}{16} \times 14^3$ | $=$ | 71.5 " |
| Total $I_B$ | $=$ | 298.6 in⁴. |

**Radii of gyration**

$$r_A = \sqrt{\frac{I_A}{A}} = \sqrt{\frac{165.2}{13.20}} = 3.54 \text{ in.}$$

$$r_B = \sqrt{\frac{I_B}{A}} = \sqrt{\frac{298.6}{13.20}} = 4.75 \text{ in.}$$

The allowable stress is $16{,}000 - 70 \times \dfrac{25.62 \times 12}{3.54} = 9{,}920$ lb. per sq. in. Considering wind, the allowable stress will be $1.50 \times 9{,}920 = 14{,}900$ lb. per sq. in. Considering dead load and total live load stresses, the unit stress is

$$S = \frac{111{,}900}{13.20} = 8{,}470 \text{ lb. per sq. in.}$$

Considering dead load, live load and impact stresses, and one-half wind load stresses.

$$S = \frac{P}{A} + \frac{M.y}{I - \dfrac{P \cdot l^2}{32E}} = \frac{115{,}300}{13.20} + \frac{16{,}750 \times 12 \times 7}{298.6 - \dfrac{115{,}300 \times 25.62^2 \times 12^2}{32 \times 30{,}000{,}000}}$$

$$= 8{,}730 + 4{,}900$$
$$= 13{,}630 \text{ lb. per sq. in.}$$

**14. Design of Portal.**—The stresses in the portal shown in Fig. 14 were calculated on the assumption that the end-posts were fixed at the base, as required in specifications, § 30.

The main diagonal members have either a tension or a compression of 10,800 lb., but the cross-section area will be governed by the compression. A section consisting of 2 angles 4″ × 3″ × $\frac{5}{16}$″ as shown in Fig. 15, will be assumed. Area = 4.18 sq. in. Assuming the angles fastened by a $\frac{1}{4}$ in. connection plate, $r_A = 0.89$ in. and $r_B = 1.88$ in. The unsupported length for axis $A$-$A$ is about 5 ft. 6 in., so $l/r = 74.2$. The unsupported length for axis $B$-$B$ is about 11 ft., so $l/r = 132/1.88 = 70$. The allowable unit stress is $1.25(16{,}000 - 70 \times 74.2) = 13{,}500$, and the required area is $10{,}800/13{,}500 = 0.80$ sq. in. The assumed angles provide 4.18 sq. in. which is more than sufficient, but will be used in order to increase the rigidity of the portal. Similar calculations show that 2 angles 4″ × 3″ × $\frac{5}{16}$″ furnish sufficient section for $U_1 U_1$. The other members carry no stress. One angle 3″ × 3″ × $\frac{1}{4}$″ will be used for these members.

**15. Design of Struts.**—The stresses in the struts are shown in Fig. 14. A section composed of 4 angles 3″ × 2½″ × ¼″, laced with 2¼ in. × ¼ in. lacing as shown in Fig. 14, will be tried. It is evident that the moment of inertia of the section about axis $B$-$B$ is smaller than about axis $A$–$A$, and will therefore control. Using ¼ in. lacing bars the angles will be ¼ in. b. to b.

$I_B = 11.7$ in.$^4$, area of section = 5.24 sq. in. and $r_B = \sqrt{\dfrac{11.7}{5.24}} = 1.48$ in., $l/r = \dfrac{17.25 \times 12}{1.48} = 140$. Specifications require that $l/r$ for laterals in this type of bridge must not exceed 150.

Allowable stress = $1.50(16{,}000 - 70 \times 140) = 9{,}300$ lb. per sq. in.

The required area is $4{,}800/9{,}300 = 0.52$ sq. in. The section assumed has an excess of area, but on account of details and to comply with specifications will be used.

**16. Design of Joints.**—All joints will be designed to develop the full strength of the members and not simply the calculated stresses. The gusset plates will be made at least thick enough to develop in bearing, the strength of the rivets in single shear. Referring to Table 33, Appendix III, this thickness is found to be $\frac{5}{16}$ in. ⅜-in. plates will be used at all joints except $L_0$ where ½-in. plates are used. The plates must be of sufficient size to contain the necessary rivets and to carry the stresses transmitted from the members.

All rivets will be made ¾ in. in diameter, except those in the flanges of channels whose depth is less than 8 in. (See Table 17, Appendix III, Maximum rivets for flanges.) The use of lug angles will be avoided wherever possible. The allowable values for rivets in shear and bearing are given in Table 33, Appendix III. The arrangement of the members at the joints is shown on

the general drawings, Fig. 18. The gage lines of angles are placed on the center line of the truss. When an angle has two gage lines, the one nearest the back is used. The center of gravity of the top chords and end-posts are placed on the center line of the truss. The size of the gusset plates is usually determined by the space required for the rivets necessary to connect the members to the

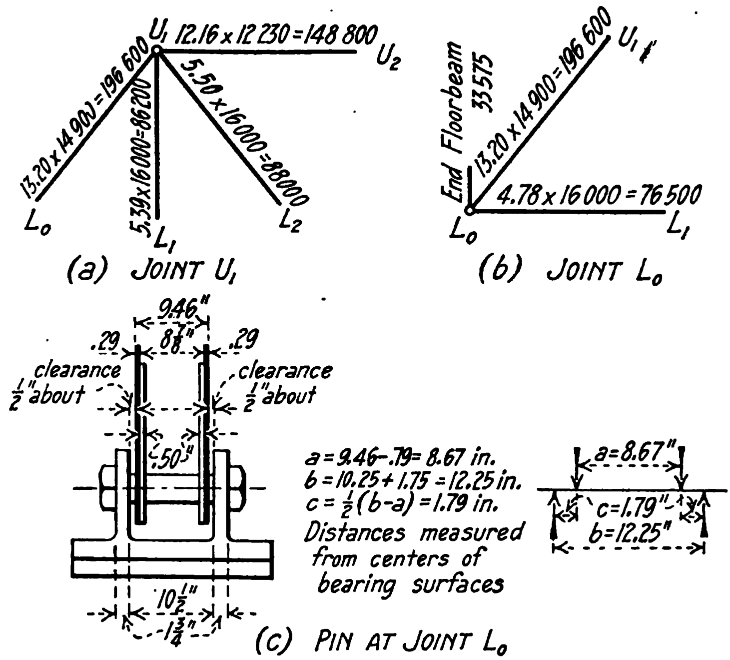



FIG. 16.

plate. Except in extreme cases, the size of the gusset plates required by the rivets will be sufficient for strength. Rivets should be symmetrically placed in order that the stress may be uniformly distributed.

17. **Joint $U_1$.**—A diagram showing the stresses in members fastened to the gusset plate at $U_1$, is given in (*a*) Fig. 16. The gusset plate will be shop riveted to $U_1U_2$ and field riveted to all other members. Bearing controls the number of rivets in $L_0U_1$. The number of field rivets required will be

$L_0U_1$
$$\frac{13.20 \times 14{,}900}{.29 \times .75 \times 20{,}000} = 45 \text{ rivets, or 23 on each side.}$$

The number of shop rivets in $U_1U_2$ at joint $U_1$ is determined by bearing, and is

$$\frac{12.16 \times 12.230}{.23 \times .75 \times 24{,}000} = 36 \text{ rivets or 18 on each side.}$$

The number of field rivets in $U_1L_2$ is determined by single shear, and is

$$\frac{5.50 \times 16{,}000}{4{,}420} = 20 \text{ rivets or 10 on each side.}$$

The number of field rivets in $U_1L_1$ is determined by single shear, and is

$$\frac{5.39 \times 16{,}000}{4{,}420} = 20 \text{ rivets or 10 on each side.}$$

18. **Joint $L_0$.**—Cast iron shoes will be used at the fixed end and cast iron rockers at the expansion end. The details of the shoes and rockers are shown in Fig. 17.

The pin at $L_0$ should be made as large as the channels of $L_0U_1$ will permit, even though a smaller pin would safely carry the stresses. A $3\frac{1}{2}$-inch pin will be used if the following investigation shows it to have sufficient strength. For the detail used here the forces acting on the pin are all vertical and have a magnitude equal to one-half the maximum pedestal reaction. This maximum will occur with the bridge fully loaded and will equal one-half the sum of the dead load, live load and impact joint loads, multiplied by the number of panels.

$$\tfrac{1}{2}(16{,}700 + 10{,}000 + 2{,}430)7 = 102{,}000 \text{ lb.}$$

The arrangement at the joint is shown in Fig. 16 and Fig. 18. The minimum thickness of the gusset plate is determined by the bearing area required, and is

$$t = \frac{51{,}000}{3\frac{1}{2} \times 24{,}000} = 0.61 \text{ in.}$$

The thickness of the web is 0.29 in., so the required thickness of the gusset plate is 0.61 − 0.29 = 0.32 in. As far as bearing is concerned, a ⅜ in. plate would be sufficient, but a ½-in. plate will be used on account of the large connection at this joint in order to give more rigidity. The maximum bending moment is

$$51{,}000 \times 1.79 = 91{,}300 \text{ in.-lb.}$$

and the maximum shear is $V = 51{,}000$ lb. The diameter required by bending moment is

$$d = \left(\frac{32M}{\pi \cdot f}\right)^{\frac{1}{3}} = 2.17\left(\frac{M}{f}\right)^{\frac{1}{3}} = 2.17\left(\frac{91{,}300}{24{,}000}\right)^{\frac{1}{3}} = 3.39 \text{ in.}$$

The diameter required by shear is

$$d = \left(\frac{4V}{\pi \cdot f_s}\right)^{\frac{1}{2}} = 1.13\left(\frac{V}{f_s}\right)^{\frac{1}{2}} = 1.13\left(\frac{51{,}000}{12{,}000}\right)^{\frac{1}{2}} = 2.33 \text{ in.}$$

The $3\frac{1}{2}$ in. pin is satisfactory, so will be used.

For bending moments on pins, see Table 27, Appendix III.

A diagram showing the stresses in the members fastened to the gusset plates at $L_0$ is given in (*b*), Fig. 16. The end-post will be shop riveted to the gusset plate, and the lower chord and floorbeam will be field riveted.

The number of shop rivets required in $L_0U_1$ at joint $L_0$ is determined by bearing on the web of the channels, and is

$$\frac{13.20 \times 14{,}900}{.29 \times .75 \times 24{,}000} = 38 \text{ or } 19 \text{ on each side.}$$

The number of field rivets in $L_0L_1$ at joint $L_0$ is determined by single shear, and is

$$\frac{4.78 \times 16{,}000}{4{,}420} = 18 \text{ or } 9 \text{ on each side.}$$

The number of field rivets required between the connection angles of the floorbeam and the gusset plate is

$$\frac{33{,}575}{4{,}420} = 7 \text{ rivets.}$$

The section area of the gusset plate required to carry the stress in $L_0L_1$ is equal to the net area of the member or 4.78 sq. in. With two gusset plates each ½ in. thick, the effective width must be $4.78/(2 \times \frac{1}{2}) = 4.78$ in. net. It is evident from the detail drawings that sufficient area has been provided. The effective section through the pin hole is more than 25 per cent of the net section of the member, and the net area behind the pin is at least 75 per cent of the area through the pin hole (§ 75).

The number of field rivets required for lateral $L_0L_1$ is

$$\frac{1.00 \times 20{,}000}{1.25 \times 3{,}750} = 4 \text{ rivets.}$$

For all other laterals the number of field rivets required is

$$\frac{0.73 \times 20{,}000}{1.25 \times 3.750} = 3 \text{ rivets.}$$

The other joints are designed in a manner similar to the method above described. The intermediate floor beams require

$$\frac{39{,}950}{4{,}420} = 9 \text{ rivets.}$$

The number of rivets at joint $U_2$ between the upper chord and the gusset plate must be sufficient to transfer to the gusset plate the maximum difference in stress between members $U_1U_2$ and $U_2U_3$. This occurs when $U_2L_3$ has its maximum stress. The difference is $10/7P \tan\theta$ + impact + difference in dead load stress $= 10/7 \times 10{,}000 \tan\theta + 0.243 \times 10/7 \times 10{,}000 \tan\theta + 13{,}400 = 11{,}400 + 2{,}800 + 13{,}400$ lb. $= 27{,}600$ lb.

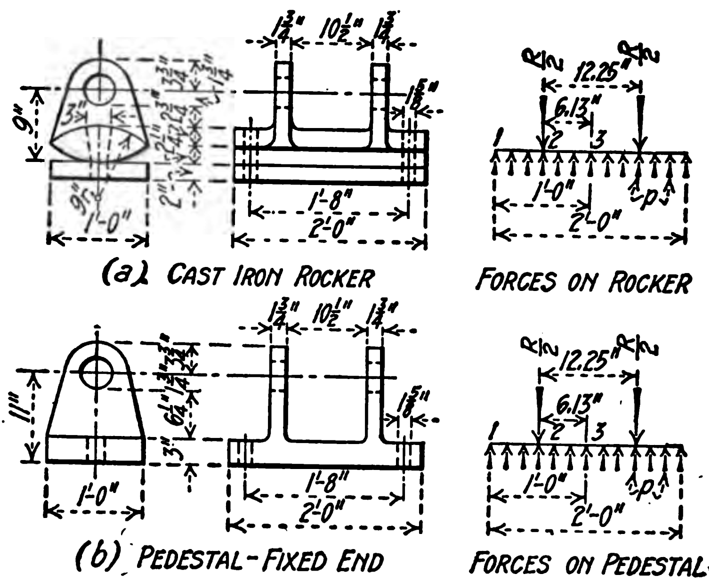



FIG. 17.

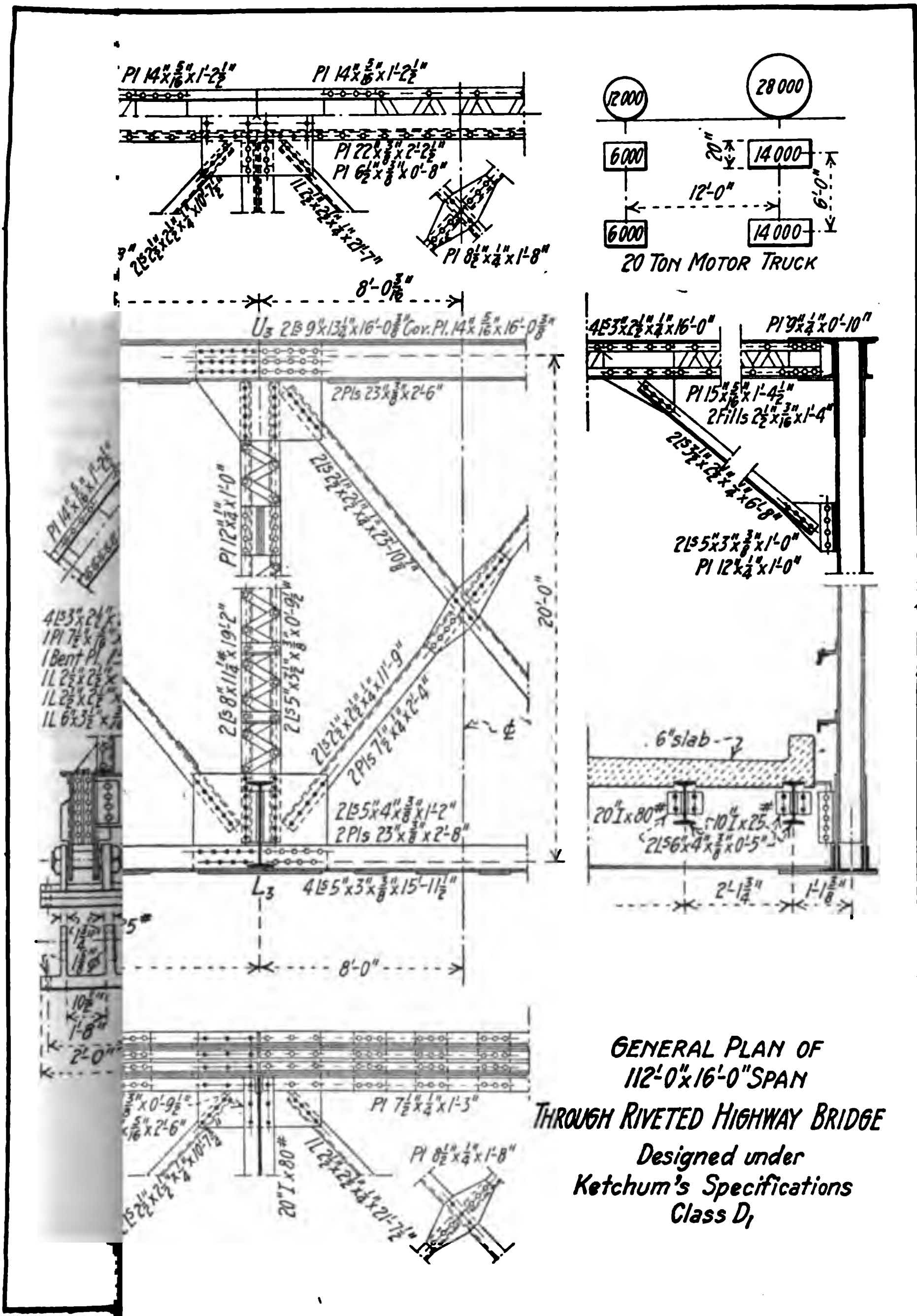
The number of shop rivets required will be determined by bearing, and is

$$\frac{27{,}600}{.23 \times .75 \times 24{,}000} = 7 \text{ rivets or 4 on each side.}$$

In order to make a rigid connection a greater number will be used.

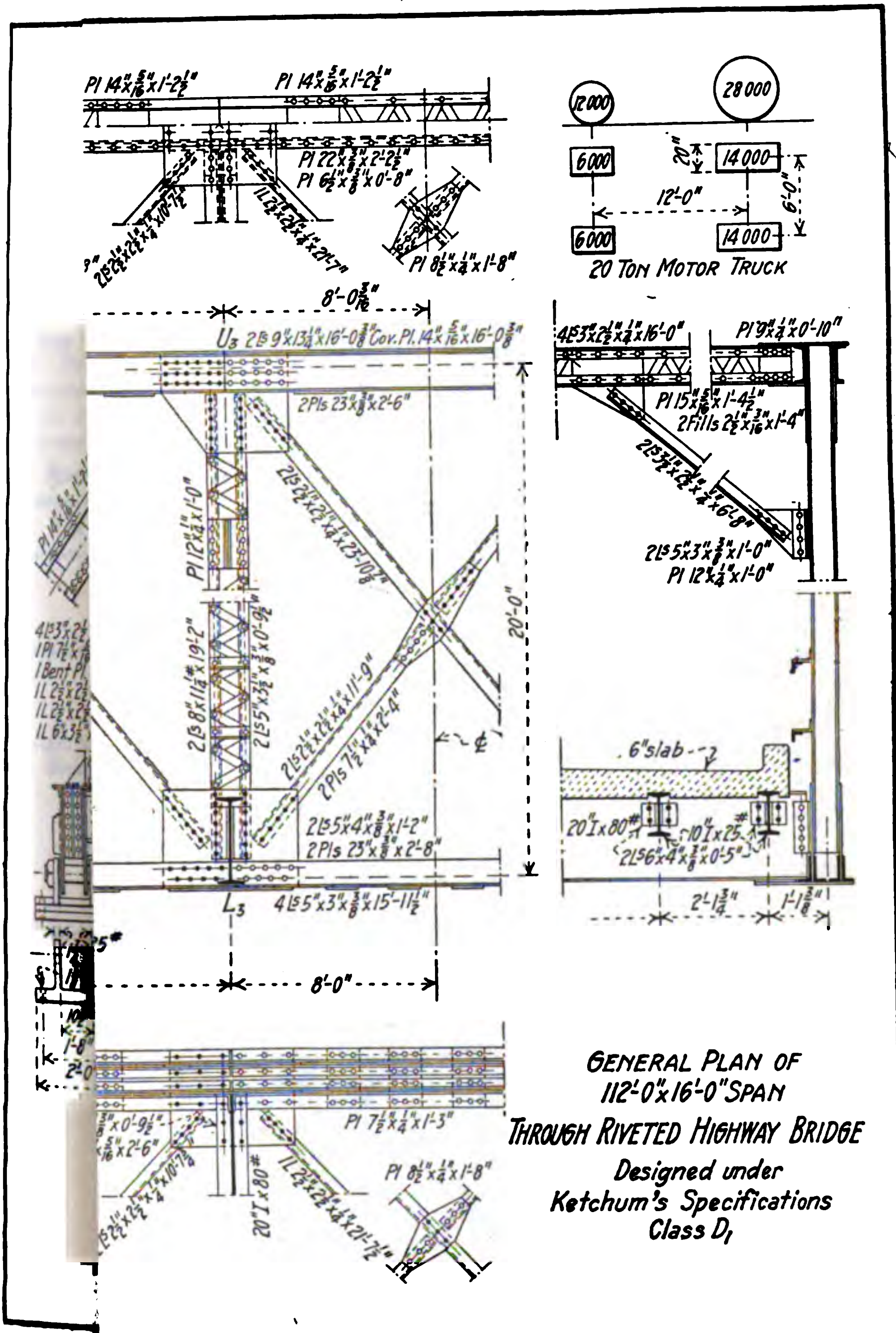
19. **End Bearings.**—Rocker or roller bearings are required on spans of 70 ft. or more, so must be used for this bridge.

20. **Design of Cast Iron Rockers.**—The type of rocker shown in Fig. 17 will be used. The maximum pedestal reaction is 102,000 lb. so the area of each masonry plate must be 102,000/600 = 170 sq. in., where 600 lb. per sq. in is the allowable bearing stress on concrete masonry. A plate 12 in. × 2 in. × 24 in. will be used. The length of the rocker will be taken as 24 in., so the bearing stress between the rocker and the plate is $p = 102{,}000/24 = 4{,}250$ lb. per lin. in. The allowable bearing stress is $300d$ pounds per lineal in., or $300 \times 18 = 5{,}400$ lb. per lineal inch.




**General Plan of 112'-0" x 16'-0" Span Through Riveted Highway Bridge**

Designed under Ketchum's Specifications Class $D_1$




GENERAL PLAN OF
112'-0" x 16'-0" SPAN
THROUGH RIVETED HIGHWAY BRIDGE
Designed under
Ketchum's Specifications
Class $D_1$

The forces acting on the rocker are shown in Fig. 17. Section 2 will be investigated as a cantilever beam with an effective length of 5 in.

$$M_2 = 4{,}250 \times (\tfrac{5}{2})^2 = 53{,}100 \text{ in.-lb.}$$

$$M_3 = 51{,}000 \times 6.13 - 4{,}250 \times (\tfrac{12}{2})^2 = 663 \text{ in.-lb.}$$

The moment of inertia of the section is 44.03 in$^4$., $c = 2\frac{1}{4}$ in., so the largest bending stress is

$$S = \frac{M \cdot c}{I} = \frac{53{,}100 \times 2.25}{44.03} = 2{,}710 \text{ lb. per sq. in.}$$

The shear to the left of section 2 is $5 \times 4{,}250 = 21{,}250$ lb., and to the right of 2 is $(12 - 6.75)4{,}250 - 51{,}000 = 28{,}700$ lb. The section area is 38.33 sq. in. so the largest average unit shear is $28{,}700/38.33 = 750$ lb. per sq. in. The depth of $4\frac{1}{2}$ in. is sufficient for bending and shear so will be used. The thickness of the upright section is determined by the bearing area on the pin. Using an allowable bearing stress of 9,000 lb. per sq. in. for cast iron, for a $3\frac{1}{2}$-in. pin, we have $2 \times 3\frac{1}{2} \times t \times 9{,}000 = 102{,}000$ or $t = 1.62$ in.; $1\frac{3}{4}$ in. will be used. The length of the upright section is short so there will be no column action.

21. **Fixed End.**—The type of pedestal shown in Fig. 17 will be used at the fixed end. The same bearing stress on the masonry exists here as at the free end. The forces acting are shown in Fig. 17. The maximum bending moment occurs at 2 and is 53,100 in.-lb. The moment of inertia of the section is $\frac{1}{12} \times 12 \times 3^3 = 27$

$$S = \frac{M \cdot c}{I} = \frac{53{,}100 \times 1\frac{1}{2}}{27} = 2{,}950 \text{ lb. per sq. in.}$$

The maximum shear is 28,700 lb., and the maximum average unit shear is

$$\frac{28{,}700}{3 \times 12} = 780 \text{ lb. per sq. in.}$$

The upright sections are the same as at the fixed end. There will be no column action.

22. **Detail Drawings.**—Detail drawings are given in Fig. 18.

# CHAPTER XV.

## Design of Steel Highway Bridge Details.

**Introduction.**—The different types of steel highway bridges have been considered in previous chapters.

**Proportions of Girders and Trusses.**—The economic depth of a girder is approximately such as to make the weight of the web including stiffeners equal to the weight of the flanges. The economic depth of a truss is such as to make the weight of web members approximately equal to the weight of the chord members. As a general rule the depth of trusses should be from $\frac{1}{5}$ to $\frac{1}{7}$ the span length. The depth of inclined chord trusses should be greater than the depth of parallel chord trusses for the same span. The height at the ends of the bridge should be sufficient for an effective portal. The width between trusses of medium span highway bridges is usually determined by the requirements of traffic. The width in no case should be less than $\frac{1}{20}$ the span. The most economic inclination of diagonals is about 40 degrees, so that in a Pratt truss the panel length should be about 0.42 times the depth of truss. Some economy in weight of steel can be gained in long span truss bridges by making the panel lengths a constant ratio of the depth. However this is rarely done for the reason that the increased cost of shop work and erection due to the increased complexity of the design usually more than offset the saving in structural steel. Increasing the panel length increases the weight of the floor, and the panel lengths should therefore be less for a heavy concrete floor than for a plank floor. The increase in weight of superstructure for permanent floors will vary with details and loads. Calculations by Mr. Clifford Older, bridge engineer, Illinois Highway Commission, shows that a variation of 10 lb. per sq. ft. in the weight of the floor makes a similar variation of about 4 per cent in the weight of the superstructure.

**Economic Span.**—In "Bridge Engineering," page 1,187, Dr. J. A. L. Waddell has shown that for a crossing of indefinite length and with piers and abutments of uniform depth, "The greatest economy will occur when the cost per lineal foot of the trusses and laterals of the superstructure is equal to the cost per lineal foot of the substructure." The floor and its supports being independent of the span is not included. This analysis assumes that all spans will be of equal length. With a stream having a deep channel near its center, the piers near the center will be much more expensive than those near the shore, and the center spans should be made longer than the shore spans.

In a paper entitled "Economic Span Length for Bridges," presented before the Western Society of Engineers, Vol. 24, No. 4, Dr. J. A. L. Waddell gives the results of a study of economic span lengths of bridges on deep foundations. For highway bridges resting on deep foundations in sand he obtains the economic span lengths as follows. For foundations 100 ft. deep, 300 ft. span for low lèvel bridges, and 325 ft. span for high level bridges. For foundations 150 ft. deep, 350 ft. span for both low level and high level bridges. For foundations 200 ft. deep, 400 ft. span for low level bridges, and 375 ft. for high level bridges.

The most important conclusions arrived at in the paper are:

1. "For all types of bridges the economic span length increases with the depth of foundations, though not necessarily in the same proportion.

2. "The lighter the superstructure and the live load it carries, the greater generally is the economic span length, and the greater the variation of the latter with the depth of foundation.

3. "Structures with piers founded on bed rock generally have economic span lengths somewhat greater than those of the corresponding structures founded on sand."

**KINDS OF STRESS.**—In addition to the stresses due to (1) dead load, (2) live or moving load, (3) wind load, and (4) snow load as calculated in Part I, it is necessary to consider; (5) impact stresses, (6) temperature stresses, (7) centrifugal stress, and (8) secondary stresses not taken into account in the calculations. In addition to the above it is necessary in determining the allowable stress in any member to take into account the imperfections in materials and workmanship, possible increase in live loads, fatigue of metals, the frequency of the application of stress, corrosion and deterioration of materials, etc. The structure should be so designed that no part under any condition shall be stressed beyond the elastic limit. The allowable stresses for dead load are usually taken at about 50 to 60 per cent of the elastic limit of the material. The live load stresses are equivalent to static load stresses plus impact stresses. If the live load stresses are increased for impact the resulting stresses may be considered as dead load stresses. With steel with an elastic limit of about 32,000 lb. per sq. in. it is the common practice to assume a safe tensile unit stress of 16,000 lb. per sq. in. for dead load stresses and for live load stresses plus impact. The allowable compressive stress would be $16,000 - 70l/r$, where $l$ = the length of the member and $r$ = the least radius of gyration of the member, both in inches, with a maximum unit stress of 14,000 lb. per sq. in. The stress in any tension or compression member due to its own weight is neglected, providing the additional stress due to weight does not increase the allowable unit stress permitted for dead load, live load and impact by more than 10 per cent. The stress in any tension or compression member due to wind load is neglected, providing the additional stress due to wind load does not increase the allowable unit stress permitted for dead load, live load and impact by more than 25 per cent. The secondary stresses may in extreme cases produce stresses as high as 40 per cent of the sum of the dead load stresses, and the live load and impact stresses. Some allowance must also be made for over load, for corrosion and for the other conditions. It is not probable that all of the additional stresses will occur when the live load is a maximum, and that the sum of all the probable stresses will always be less than 32,000 lb. per sq. in.—the elastic limit of the material. A bridge designed for a unit stress of 16,000 lb. per sq. in. in tension and with an allowable stress in compression of $16,000 - 70l/r$ lb. per sq. in. is said to be designed with a factor of safety of four. The factor of safety is the number by which we divide the ultimate strength of the member to obtain the working stress. A factor of safety of four in this case means that the structure is just as strong as it should be, and not four times as strong as it should be, as is sometimes assumed.

**IMPACT STRESSES.**—As a load moves over the bridge it causes shocks and vibrations whereby the actual stresses are increased over those due to static loads alone. It is shown in mechanics of materials that a load suddenly applied to a bar or a beam will produce stresses equal to twice the stresses produced by the same load gradually applied. A bridge is a complex structure and it is not possible to determine the exact effect of the moving loads. It has been found by experiment that the ultimate strength for repeated loads is much less than the ordinary ultimate strength. In a bridge it will be seen that the dead load is a fixed load and that the live load is a varying load.

For stresses of one kind Professor Launhardt has proposed the following formula:

$$P = S\left(1 + \frac{\text{Min. stress}}{\text{Max. stress}}\right) \tag{1}$$

where $P$ is the allowable working stress required, and $S$ is the allowable working stress for live loads, varying from zero to the maximum stress. For stresses of opposite kinds Professor Weyrauch has proposed the following formula:

$$P = S\left(1 - \frac{\text{Min. stress}}{2\ \text{Max. stress}}\right) \tag{2}$$

where $P$ and $S$ are the same as for the Launhardt formula, the maximum and minimum stresses being taken without sign. For columns and struts the allowable stresses are to be reduced by a suitable column formula.

There are three methods in common use for taking account of impact: (1) Impact formulas; (2) Launhardt-Weyrauch formulas, and (3) Cooper's Method.

(1) **Impact Formulas.**—The formula in most common use is given in the form

$$I = S\left(\frac{a}{L + b}\right) \tag{3}$$

where $I$ = impact stress to be added to the static live load stress, $S$ = the static live load stress, $L$ = the length in feet of the portion of the bridge that is loaded to produce the maximum stress in the member, and $a$ and $b$ are constants expressed in feet. The American Railway Engineering Association specify for railway bridges, $a = b = 300$ ft.

After an extensive series of tests the committee on Iron and Steel Structures of the American Railway Engineering Association has recommended the following formula for impact in railway bridges.

$$I = S \cdot \frac{30{,}000}{30{,}000 + L^2} \tag{4}$$

where $L$ is the length of the span of the bridge. This formula has not as yet (1919) been adopted by the Association.

Mr. C. C. Schneider, M. Am. Soc. C. E., specified that for electric railway bridges

$$I = S \cdot 150/(L + 300) \tag{5}$$

In the Osborn Engineering Company's 1903 specifications for railway and for highway bridges the impact is calculated by the formula

$$I = S \cdot S/(S + D) \tag{6}$$

where $S$ is the static live load stress and $D$ is the dead load stress. This method is also specified by the Harriman Railway System.

(2) **Launhardt-Weyrauch Formulas.**—Formula (1) is used for determining the allowable stress for loads of one kind, and formula (2) is used for determining the allowable stress for loads of different kinds. This method is used in Thacher's Specifications, and others.

(3) **Cooper's Method.**—Cooper uses formula (1) and calculates the area for the dead load and the area for the live load stress separately. For dead loads from formula (1) we have $P = 2S$, while for live loads the range of stress is from zero to the maximum, and $P = S$.

For a reversal of stress Cooper designs the member to take both kinds of stress, but to each stress he adds eight-tenths of the lesser of the two stresses.

The different methods, while apparently very unlike, give essentially the same results. Many engineers claim that fatigue of the materials and impact should be considered separately. In choosing working stresses, an allowance should be made for secondary and other stresses that cannot easily be calculated, for corrosion, etc.

For the recent practice in the use of impact in the design of highway bridges, see Chapter IX.

**Temperature Stresses.**—An increase or decrease in temperature produces no stresses in a bridge with one end on frictionless rollers. Where there is a horizontal resistance to the movement, the bridge becomes a two-hinged arch and it is necessary to calculate the stresses for that case. See the author's "Steel Mill Buildings," Chapter XIV.

**Centrifugal Stresses.**—Mr. C. C. Schneider's "Specifications for Electric Railway Bridges" contains the following requirement:

Structures located on curves shall be designed for the centrifugal force of the live load acting at the top of the rail. The centrifugal force shall be calculated by the following formula:

$$C = (0.043 - 0.003D)W \cdot D \tag{7}$$

where $C$ = centrifugal force in lb.;
$W$ = weight of train in lb.;
$D$ = degree of curvature.

**SPECIFICATIONS FOR STEEL.**—All standard specifications call for open hearth steel. It has been the custom to specify "soft" steel with an ultimate strength of, say, 54,000 to 62,000 lb. per sq. in.; "medium" steel with an ultimate strength of, say, 60,000 to 68,000 lb. per sq. in.; and "rivet" steel with an ultimate strength of, say, 50,000 to 58,000 lb. per sq. in. The American Railway Engineering Association has specified a single grade of "structural" steel with an ultimate strength of 55,000 to 65,000 lb. per sq. in.; and "rivet" steel with an ultimate strength of 46,000 to 56,000 lb. per sq. in. This method appears to be coming rapidly into use and promises to become standard.

Standard specifications are given in Appendix I, in which the specifications for material adopted by the American Society for Testing Materials have been used.

**ALLOWABLE STRESSES.**—The allowable stresses in the different members of steel highway bridges will depend upon the method of providing for impact.

**Schneider's Specifications.**—In his "Specifications for Steel Electric Railway Bridges" Mr. C. C. Schneider has specified the allowable stresses adopted by the American Railway Engineering Association using the impact formula in (5). The clauses referring to unit stresses are as follows:

§ 17. *Unit Stresses.*—All parts of structures shall be so proportioned that the sum of the maximum stresses shall not exceed the following amounts in lb. per sq. in. except as modified in paragraphs 25 to 27.

§ 18. *Tension.*—Axial tension on net section . . . . . 16,000

§ 19. *Compression.*—Axial compression on gross section $16{,}000-70.l/r$ where $l$ is the length and $r$ is the least radius of gyration of member, both in inches.

§ 20. *Bending*: on extreme fibers of rolled shapes, built sections and girders; net section 16,000
On extreme fibers of pins . . . . . 24,000

§ 21. *Shearing*: shop driven rivets and pins . . . . . 12,000
Field driven rivets and turned bolts . . . . . 10,000
Plate girder webs; gross section . . . . . 10,000

§ 22. *Bearing*: shop driven rivets and pins . . . . . 24,000
Field driven rivets and turned bolts . . . . . 20,000
Granite masonry and Portland cement concrete . . . . . 600
Sandstone and limestone . . . . . 400
Expansion rollers; per linear inch . . . . . 600 $d$
Where $d$ is the diameter of the roller in inches.

§ 23. *Limiting Length of Compression Members.*—No compression member shall have a length exceeding 100 times its least radius of gyration, excepting those for wind bracing, which may have a length 120 times the least radius of gyration.

§ 24. *Alternate Stresses.*—Members subject to alternate stresses of tension and compression shall be proportioned for the stresses giving the largest section. If the alternate stresses occur in succession during the passage of one train, as in stiff counters, each stress shall be increased by 50 per cent of the smaller. The connections shall in all cases be proportioned for the sum of the stresses.

§ 25. *Counters.*—Wherever the live and dead load stresses are of opposite character, only 70 per cent of the dead load stress shall be considered as effective in counteracting the live load stress.

§ 26. *Combined Stresses.*—Members subject to the action of both axial and bending stresses shall be proportioned so that the combined stress shall not exceed the allowed axial stress.

§ 27. *Lateral and Other Stresses Combined.*—For stresses produced by lateral and wind forces combined with those of live loads, dead loads and centrifugal forces, the unit stresses may be increased 25 per cent over those given above; but the section shall not be less than required if lateral and wind forces be neglected.

**Cooper's Specifications.**—In his 1909 "Specifications for Steel Highway and Electric Railway Bridges," Mr. Theodore Cooper specified the following allowable unit stresses:

*Tension. Medium Steel.*—Floorbeam hangers and other similar members liable to sudden loading, net section 8,000 lb. per sq. in.; longitudinal, lateral and sway bracing for wind and live load stresses, 18,000 lb. per sq. in.; solid rolled beams used as cross floorbeams and stringers, 13,000 lb. per sq. in.; bottom flanges of riveted girders, net section, all moment resisted by flanges, bottom chords, main diagonals, counters 12,500 lb. per sq. in. for live load stress, and 25,000 lb. per sq. in. for dead load stress. Verticals carrying floorbeams, 10,000 lb. per sq. in. for live load stress, and 20,000 lb. per sq. in. for dead load stress.

Soft steel may be used with tensile unit stresses 10 per cent less than above, execpt for eye-bars.

*Compression. Medium Steel.*—For chord segments $P = 12,000 - 55 \cdot l/r$ lb. per sq. in. for live load stresses, and $P = 24,000 - 110 \cdot l/r$ lb. per sq. in. for dead load stresses.

For all posts for through bridges, including end-posts, $P = 10,000 - 45 \cdot l/r$ lb. per sq. in. for live load stresses, and $P = 20,000 - 90 \cdot l/r$ lb. per sq. in. for dead load stresses.

For all posts in deck bridges and trestles, $P = 11,000 - 40 \cdot l/r$ lb. per sq. in. for live load stresses, and $P = 22,000 - 80 \cdot l/r$ lb. per sq. in. for dead load stresses.

For lateral struts and rigid bracing, $P = 13,000 - 60\ l/r$ lb. per sq. in for wind stresses and ¾ of the above for live load stresses.

In above $l$ = length of member $c$ to $c$ of connections, and $r$ = least radius of gyration of member, both in inches. The ratio of $l/r$ shall not exceed 100 for main members and 120 for laterals.

Soft steel may be used with unit stresses 10 per cent less than the above, except for eye-bars.

*Bending.*—The bending on extreme fibers of pins shall not exceed 20,000 lb. per sq. in.

*Shear.*—The shear on pins and rivets in trusses shall not exceed 10,000 lb. per sq. in. The shear on rivets in floor systems shall not exceed 80 per cent, and in lateral systems shall not exceed 150 per cent of the value above.

*Bearing.*—The bearing on pins and rivets in trusses shall not exceed 15,000 lb. per sq. in. for live load and 30,000 lb. per sq. in. for dead load. The bearing on rivets in floor systems shall not exceed 80 per cent, and in lateral systems shall not exceed 150 per cent of the values above.

Field rivets shall have their allowable bearing and shear reduced one-third.

Bearing on rollers per lineal inch = $300d$, where $d$ is the diameter of the roller in inches.

Bearing on masonry shall not exceed 250 pounds per sq. in.

*Combined Stresses.*—Members subject to combined stresses must be designed for the greatest stress. Unless the stress due to weight, only, exceeds 10 per cent of the allowed stress, such stress need not be considered. Unless the stress due to wind forces exceeds 30 per cent of the allowed unit stress it need not be considered.

*Reversal of Stress.*—Members and their connections subject to alternate stress shall be designed to take each kind of stress. Both stresses, shall however, be increased by an amount equal to 8/10 of the least of the two stresses.

**Engineering Institute of Canada.**—General Specification for Steel Highway Bridges adopted 1918 by the Engineering Institute of Canada specifies unit stresses in lb. per sq. in. as follows: Axial tension on net section of steel, 16,000. Axial compression on gross section of columns, $12,000 - 0.3(l/r)^2$, in which $l$ = length in inches, and $r$ = least radius of gyration in inches. Direct compression on steel castings, 14,000. Direct compression on iron castings, 10,000. Bending on extreme fibers of rolled shapes, built-up sections and girders, net section 16,000; steel castings 12,000; iron castings, 3,000; pins 24,000; white oak, Douglas fir and southern long leaf pine, 1,600, white and red pine, 1,100. Shearing on power driven shop rivets and pins, 11,000; power driven field rivets, 10,000; hand driven field rivets and turned bolts, 8,000; plate girders, gross section, 10,000. Bearing on power driven shop rivets 22,000; power driven field rivets and pins, 20,000; hand driven field rivets and turned bolts, 16,000; hard bronze expansion bearings, 1,000; expansion rollers, per lineal inch, 600 $d$, where $d$ = diameter of roller in inches; granite masonry, 800, concrete 1 : 2 : 4 mix, 600, limestone masonry, 400; sandstone masonry 300. Wind load stresses need not be considered unless in combination with dead and live load stresses the stresses exceed the allowable stresses for dead and live load by more than 25 per cent. Stresses due to weight of member or eccentric loading need not be considered unless in combination with dead and live load stresses the stresses exceed the allowable stresses for dead and live loads by more than 10 per cent. When dead and live load stresses are of opposite character, only two-thirds of the minimum dead load stress shall be considered as effective in counteracting live load stress. If reversal is due to moving load, each kind of stress shall be increased by 50 per cent of the smaller.

**Illinois Highway Commission.**—The following allowable stresses are specified. No allowance is made for impact.

*Tension.*—Medium steel and steel castings, 16,000 lb. per sq. in.

*Compression.*—Medium steel, $16,000 - 70(l/r)$ lb. per sq. in., but not to exceed 14,000 lb. per sq. in., where $l$ = length of member and $r$ = radius of gyration of member, both in inches.

Cast Steel, 16,000 lb. per sq. in.

*Bending.*—Extreme fiber stress on rolled and built up sections and steel castings, 16,000 lb. per sq. in. Extreme fiber stress on pins, 24,000 lb. per sq. in.

*Shear.*—Shop rivets and pins, 10,000 lb. per sq. in. Bolts and field rivets, 8,000 lb. per sq. in. Webs of rolled and built section (average) 10,000 lb. per sq. in.

*Bearing.*—Pins and shop rivets, 20,000 lb. per sq. in. Bolts and field rivets, 16,000 lb. per sq. in. Expansion rollers (steel rollers on steel plates), $600d$ lb. per lineal inch, where $d$ = diameter of roller in inches. Expansion rockers (cast iron, State standard), $300d$ lb. per lineal inch.

*Reversal of Stress.*—Connections of members carrying reversing stresses shall be proportioned for a stress found by adding ¾ of the lesser to the greater stress.

*Wind Stress.*—Allowable stresses in chords may be increased 25 per cent to provide for wind loads.

**Iowa Highway Commission.**—The following stresses on steel in lb. per sq. in. are specified:

*Tension.*—Axial tension on net section .......... 16,000

*Compression.*—Axial compression on gross section 16,000 − 70 $l/r$ where $l$ is the unsupported length of the member in inches and $r$ is the least radius of gyration in inches.

*Bending.*—On extreme fibers of rolled shapes, built sections and girders; net section . 16,000
On extreme fibers of pins, rivets and bolts .......... 25,000
*Shearing.*—On pins and shop driven rivets .......... 12,000
On field driven rivets and turned bolts .......... 9,000
On plate girder web; gross section .......... 10,000
*Bearing.*—On pins and shop driven rivets .......... 24,000
On field driven rivets and turned bolts .......... 18,000
On masonry .......... 400
On steel expansion rollers or rockers where $d$ is the diameter of the rocker or roller in inches, per linear inch .......... 600 $d$
On pin bearing on rockers .......... 12,000
*For Cast Steel.*—Tension .......... 16,000
Compression .......... 16,000
Shear .......... 10,000

*Alternate Stresses.*—Members subject to alternate stress of tension and compression shall be proportioned for the stress giving the largest section. If the alternate stresses occur in succession during the passage of one load, each stress shall be increased by fifty (50) per cent of the other. The connections shall in all cases be proportioned for the sum of the stresses.

*Counter Stresses.*—Wherever live and dead load stresses are of opposite character, only 70 per cent of the dead load stresss shall be considered as effective in counteracting the live load stress.

*Axial and Bending Stresses Combined.*—Members subject to both axial and bending stresses shall be proportioned so that the combined fiber stresses will not exceed the allowed axial stress.

*Lateral and Other Stresses Combined.*—For stresses produced by lateral or wind forces combined with those from live and dead load forces, the unit stress may be increased 30 per cent over those given above; but the section shall not be less than required if the lateral or wind forces be neglected.

**MINIMUM THICKNESS OF METAL.**—Illinois Highway Commission specifies that the minimum thickness of metal shall be 5/16 in. except for fillers. Gusset plates shall not be less than ⅜ in. thick. Webs of beams and channels shall not be less than ¼ in.

Iowa Highway Commission specifies that the minimum thickness of metal shall be ¼ in. except for webs of channels and fillers.

Cooper's Specifications (1909) require that the minimum thickness of main members and their connections be 5/16 in., and for laterals and their connections be ¼ in., except for fillers.

The author recommends that 5/16 in. be the minimum thickness except for $D_1$ and $D_2$ bridges where the minimum shall be ¼ in. except for webs of channels, which may be 0.20 in. in thickness.

The standard minimum thickness for metal in railway bridges is ⅜ in.

**TENSION MEMBERS.**—Tension members are made (1) of eye-bars; (2) of square or round loop bars; (3) of simple shapes, and (4) of built sections.

**Eye-bars.**—Eye-bars are used for main tension members of pin-connected trusses. The eyes may be formed (*a*) by upsetting and forging, or (*b*) by piling and welding. By the first method the bar is upset and the head is forged in a die, after which the bar is reheated and annealed and the pin hole is drilled. By the second method a "pile" of iron bars is placed on the end of the bar, the pile is heated and the head is forged in a die. The bar is then reheated and annealed and the pin hole is drilled. Steel eye-bars should always be made by upsetting and forging. The American Bridge Company's standard eye-bars are given in Fig. 1 and in Table 20, Appendix III. Eye-bars thinner than those specified are liable to buckle in the head. Eye-bars may be obtained in different thicknesses varying by 1/16 inch. Eye-bars are seldom made with a thickness of more than one-third or less than one-sixth of the depth of the bar. The Osborn Engineering Company specifies that bars shall not be less than ¾ in. in thickness, and preferably not less in

thickness than ⅛ the depth. Eye-bars should be parallel as nearly as possible; the maximum variation should never be greater than one inch in eight feet. The specifications in Appendix I require that eye-bars shall not be out of line more than one inch in 16 feet (§ 92). Thick bars give

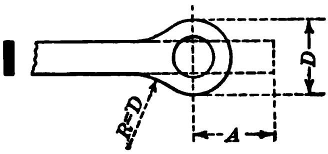



FIG. 1. ORDINARY EYE-BARS.

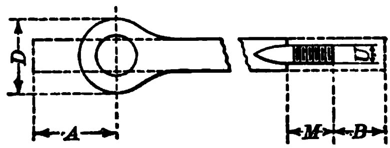



FIG. 2. ADJUSTABLE EYE-BARS.

large moments on the pin. Pins are ordinarily specified to be not less than three-fourths of the depth of the deepest bar coming on the pin. Bars very shallow or very deep will therefore require large pins. The stresses in eye-bars due to their own weight are given in Fig. 4, Chapter VI. Eye-bars should always be used in pairs and should be kept small in order to keep down the size of the pins and reduce the cost of fabrication of the pins. Specifications for eye-bars are given in Appendix I.

**Adjustable Eye-bars.**—Where eye-bars are used for counters they are made adjustable. The American Bridge Company's standard adjustable eye-bars are given in Fig. 2, and in Table 20, Appendix III. The parts of the bar may be connected by sleeve nuts or turnbuckles.

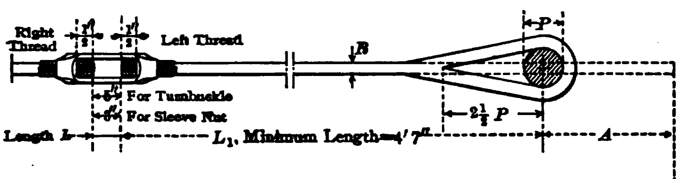



FIG. 3. LOOP-BAR.

**Loop-bars.**—Iron bars, both square and round, are often made with loop ends. Steel bars should never be used with loop ends for the reason that welded steel is not ordinarily considered reliable. The American Bridge Company's standard loop-bars are shown in Fig. 3, and in Table 21, Appendix III. Loop-bars are made with both single and double loops. Clevises are to be preferred to double loops. Loop-bars bent in the weld should not be used.

**Standard Upsets.**—Bars upon which screw ends are to be cut, are first upset so that the area through the base of the screw will be in excess of the main body of the bar by a required amount, varying from 16 to 40 per cent. The American Bridge Company's standard upsets for round and square bars are given in Table 18 and Table 19, Appendix III.

**Clevises.**—Where small round or square steel bars are used, the ends should be upset and the connection to the pin should be made by means of clevises. The American Bridge Company's standard clevises are given in Fig. 4, and in Table 22, Appendix III.

**Turnbuckles and Sleeve Nuts.**—Eye- or loop-bars are made adjustable by means of turnbuckles or sleeve nuts. Turnbuckles are more often used than sleeve nuts. The turnbuckle has the advantage that the ends of the bars are visible, while it has the disadvantage that is can be loosened with a bar. The American Bridge Company's standard turnbuckles and sleeve nuts are given in Fig. 5 and Fig. 6, and in Table 23, Appendix III.

**Riveted Tension Members.**—The problem in the design of riveted tension members is the design of the end connections. The rivets in the end connections should be symmetrical with the

neutral axis of the member. This is sometimes difficult to attain, and results in large eccentric stresses. In riveted tension members with pin-connections it is usually specified: (1) That the net area through the pin hole must exceed the required net area of the member by 25 per cent, and (2) the area back of the pin hole on a plane through the center of the pin hole and parallel to the axis of the member must be not less than 75 per cent of the area through the pin hole. The

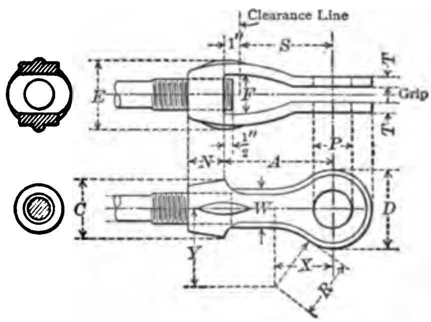



FIG. 4. CLEVIS.

net area of the member must be used in calculating the strength of a riveted tension member. In deducting for rivet holes in tension members it is often specified that rupture will be considered equally probable on a transverse or diagonal section, unless the diagonal section has a net area 30 per cent in excess of the transverse section. See § 44 in Specifications in Appendix I for the method of calculating net area of a riveted tension member.

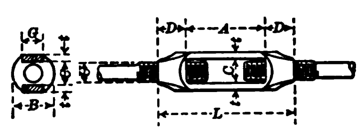


FIG. 5. TURNBUCKLE.

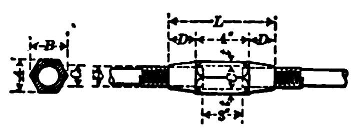


FIG. 6. SLEEVE NUT.

The net area of a tension member, $A$, required to carry a direct tension, $T$, with a safe unit stress, $f$, is $A = T/f$. For methods of calculating the stresses in tension members due to direct and cross-bending forces, see Chapter VI.

For the calculation of the stresses in an eccentric riveted connection, see Chapter VI.

The areas to be deducted for rivet holes in tension members are given in Table 35, Appendix III.

**COMPRESSION MEMBERS.**—Some of the common forms of compression members are shown in Fig. 7. The section in (*b*) consisting of two channels and a top cover plate with lacing on the bottom, and section (*e*) consisting of two channels laced on both top and bottom are commonly used for the top chords of high truss pin-connected highway bridges. Sections (*a*), (*c*), (*d*) and (*g*) are commonly used for long span highway and railway pin-connected bridges. Sections (*e*), (*f*), (*g*) and (*j*) are used for intermediate posts, while sections (*h*) to (*k*) are used for the chords of riveted highway bridges. A type of chord should be selected that will give the desired results and will at the same time give a low cost for fabrication. Where chords are made without a top cover plate, the lacing must be designed to carry the diagonal shear in addition to the usual stresses. The least radius of gyration of sections (*a*) to (*d*), inclusive, is approximately four-tenths of the width of the member, while the least radius of gyration of sections (*e*) to (*g*), inclusive, is approximately three-eighths of the depth. Approximate radii of gyration of built sections are given in Table 43, Appendix III.

In selecting a chord section the radius of gyration should be kept as large as possible, the member at the same time satisfying the following requirements: (1) The thickness of the top cover plate should not be less than 1/40 the distance between the centers of the rivets connecting the plate to the angles or channels. (2) The thickness of the side plates should not be less than

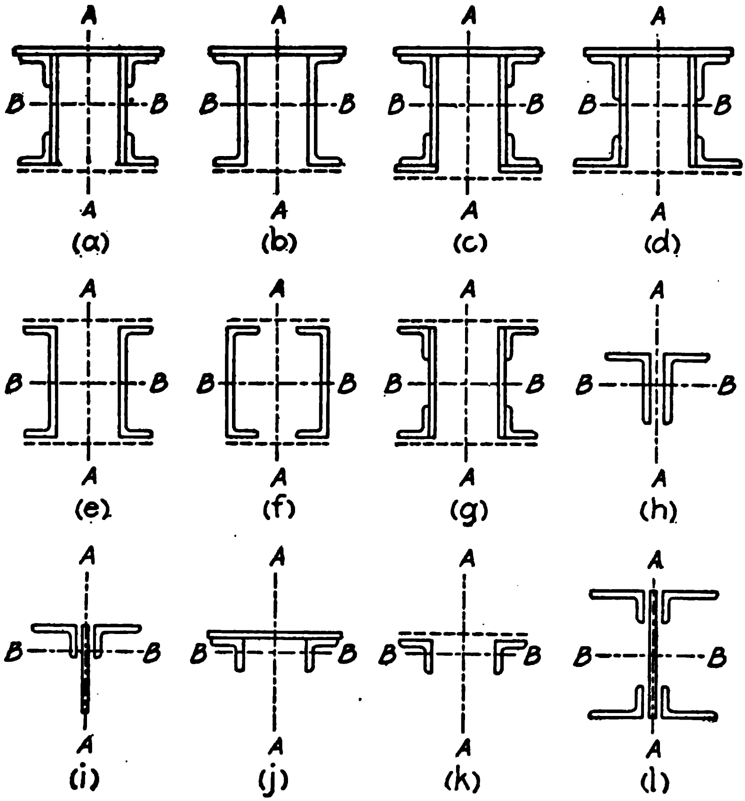
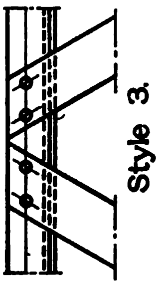



FIG. 7. RIVETED SECTIONS FOR COMPRESSION MEMBERS.

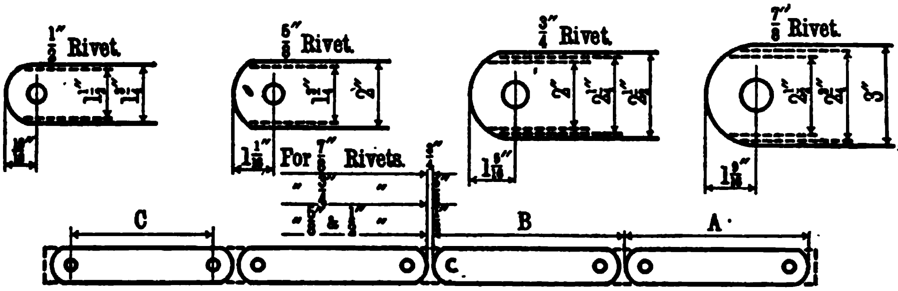
1/30 the distance between the centers of the rivets connecting it to the angles. (3) The angles should not be thinner than three-fourths the thickness of the thickest plate attached to them. (4) The radius of gyration of the member about both axes should be approximately the same. Areas, moments of inertia, radii of gyration, eccentricities and other data for built chord sections are given in the tables in Appendix III.

**DESIGN OF COMPRESSION MEMBERS.**—The allowable stresses in compression members are given in the standard specifications in this chapter. For the details of the calculations of the moments of inertia, radii of gyration and allowable stresses in compression members, see Chapter VI.

**Lacing.**—Lacing bars are used to join the parts of the member together, and make it act as a solid member to resist the shear due to bending and the diagonal shear in the member. Lacing bars are commonly made with a thickness of not less than 1/40 the distance between end rivets for single lacing, or 1/60 of the distance between rivets for double lacing riveted in the middle. The spacing should be such that the part of the column between the rivets is stronger than the


| Maximum Distance C in feet and inches for given thickness T of bar | | | | |
|---|---|---|---|---|
| T Thickness of Bar | Single Lacing | | Double Lacing | |
| | C=40 T | C=50 T | C=60 T | C=75 T |
| ⅝″ | 2′- 1″ | 2′-7¼″ | 3′- 1½″ | 3′-10⅞″ |
| 9/16 | 1-10½ | 2-4⅛ | 2- 9¾ | 3- 6 3/16 |
| ½ | 1- 8 | 2-1 | 2- 6 | 3- 1½ |
| 7/16 | 1- 5½ | 1-9⅞ | 2- 2¼ | 2- 8 13/16 |
| ⅜ | 1- 3 | 1-6¾ | 1-10½ | 2- 4⅛ |
| 5/16 | 1- 0½ | 1-3⅝ | 1- 6¾ | 1-11 7/16 |
| ¼ | 10″ | 1-0½ | 1- 3 | 1- 6¾ |




| Length to be added to Distance C | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Width of Bar in Inches | For finished length A | | | | For ordered length B | | | |
| | ½″ | ⅝″ | ¾″ | ⅞″ | ½″ | ⅝″ | ¾″ | ⅞″ |
| 3 | | | | 3⅛″ | | | | 3⅝″ |
| 2¾ | | | | 3⅛ | | | | 3⅝ |
| 2½ | | | 2⅝″ | 3⅛ | | | 3¼″ | 3⅝ |
| 2¼ | | | 2⅝ | | | | 3¼ | |
| 2 | | 2⅛″ | 2⅝ | | | 2¾″ | 3¼ | |
| 1¾ | 1⅞″ | 2⅛ | | | 2⅛″ | 2¾ | | |
| 1½ | 1⅞ | | | | 2⅛ | | | |

FIG. 8. STANDARDS FOR LACING BARS. AMERICAN BRIDGE COMPANY.

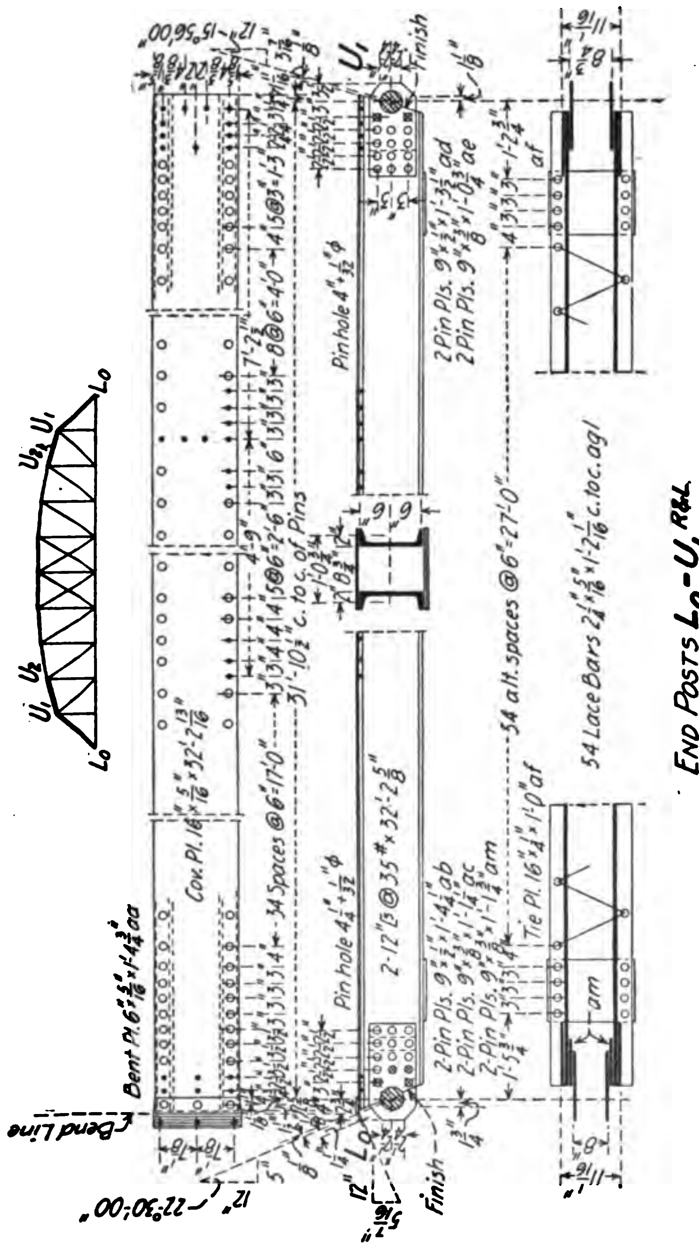



FIG. 9. DETAIL SHOP PLAN OF END-POST.

column as a whole. Specifications require that the lacing bars make an angle with the axis of the member of from 60 to 45 degrees. The American Bridge Company's standard lacing bars are given in Fig. 8.

*Design of Lacing Bars.*—The lacing bars in a column hold the parts of the column in line, carry part of the diagonal shear, and transfer part of the stress in columns with an eccentric loading. The stresses in the bars required to hold the parts of the column in line are small for stresses in the column within the elastic limit of the material. The maximum diagonal shear, $S$, in a solid member is $S = \frac{1}{2}P$, where $P$ = the total direct axial load on the member. In columns composed

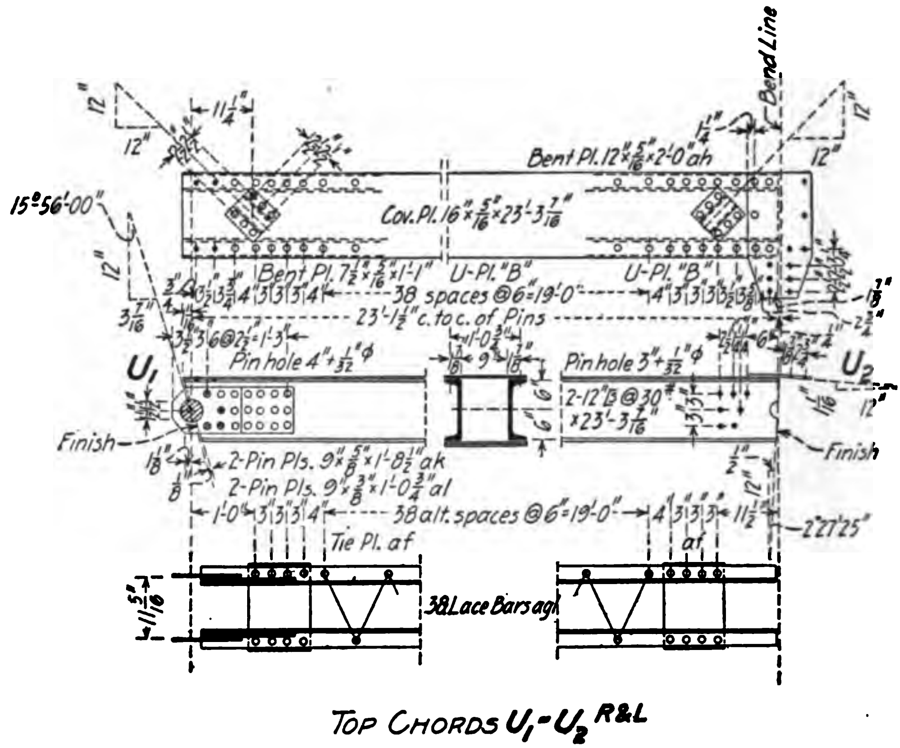



FIG. 10. DETAIL SHOP PLAN OF TOP CHORD.

of channels, angles, etc., the flange area is ordinarily sufficient to carry this shear without producing large stresses in the lacing bars. The moment, $M'$, due to the eccentric loading is $M' = P \cdot e$, where $P$ = the total direct load on the column and $e$ = the eccentricity of the loading. The lacing bars will take the shear due to this bending moment, if the flanges are light. It will be seen from the foregoing that the stresses in lacing bars depend (1) upon the make-up of the column, (2) upon the care used in building the column, and (3) upon the eccentricity of the loading.

For a column with a concentric loading, experiments show that the allowable unit stress may be represented by the straight line formula $P = 16{,}000 - 70l/r$ lb. per sq. in., where $P$ = allowable unit stress in the member, $l$ = length of the member, $c$ to $c$ of end connections, and $r$ = radius of gyration of the column, both in inches. Now the allowable unit stress on a short block is 16,000 lb., and the $70l/r$ represents the increase in the fiber stress in the column. Now if we assume that this fiber stress is caused by a horizontal load, $W$, applied at the center of the height

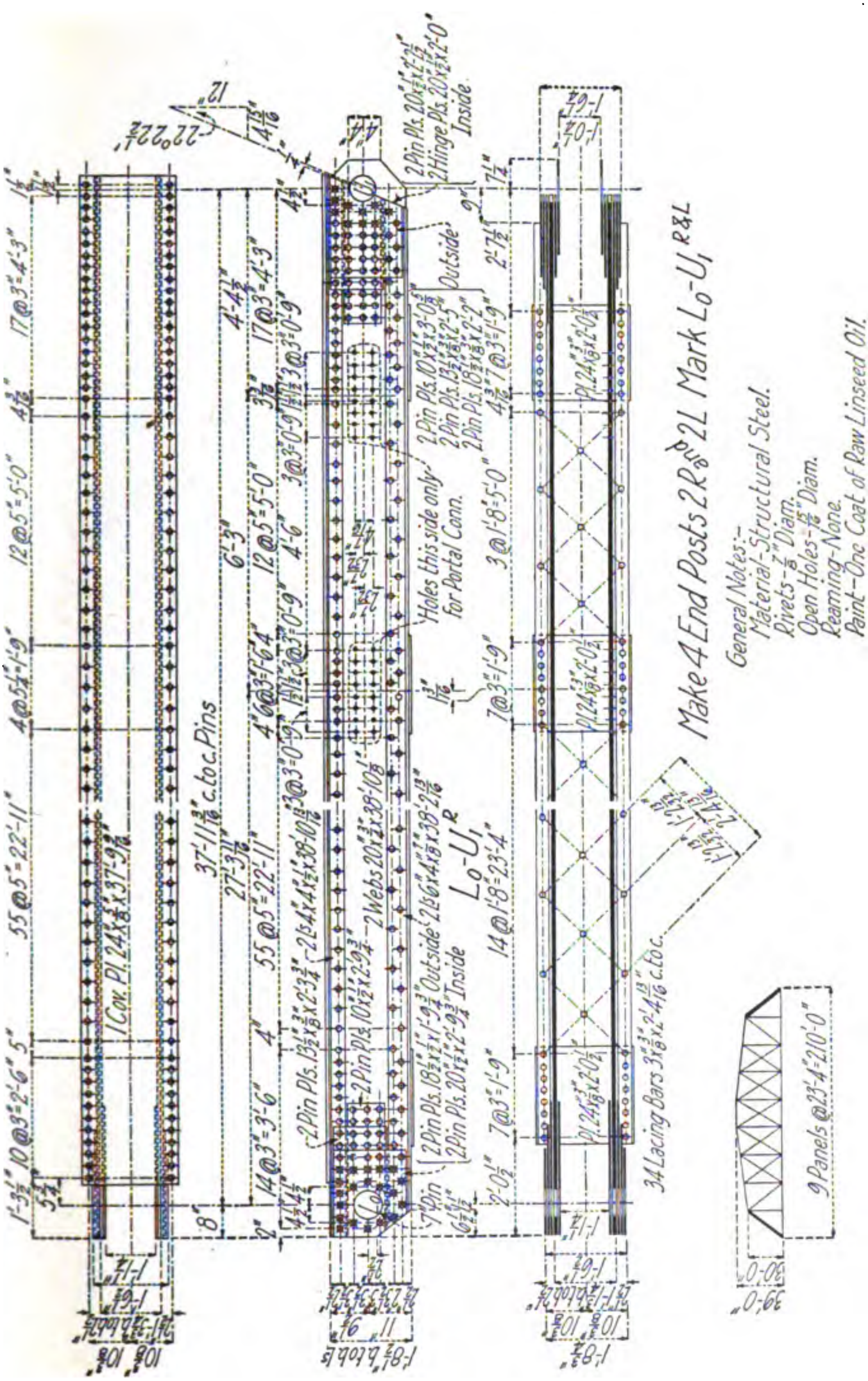



FIG. 11. SHOP DRAWING OF END-POST OF PIN-CONNECTED TRUSS.

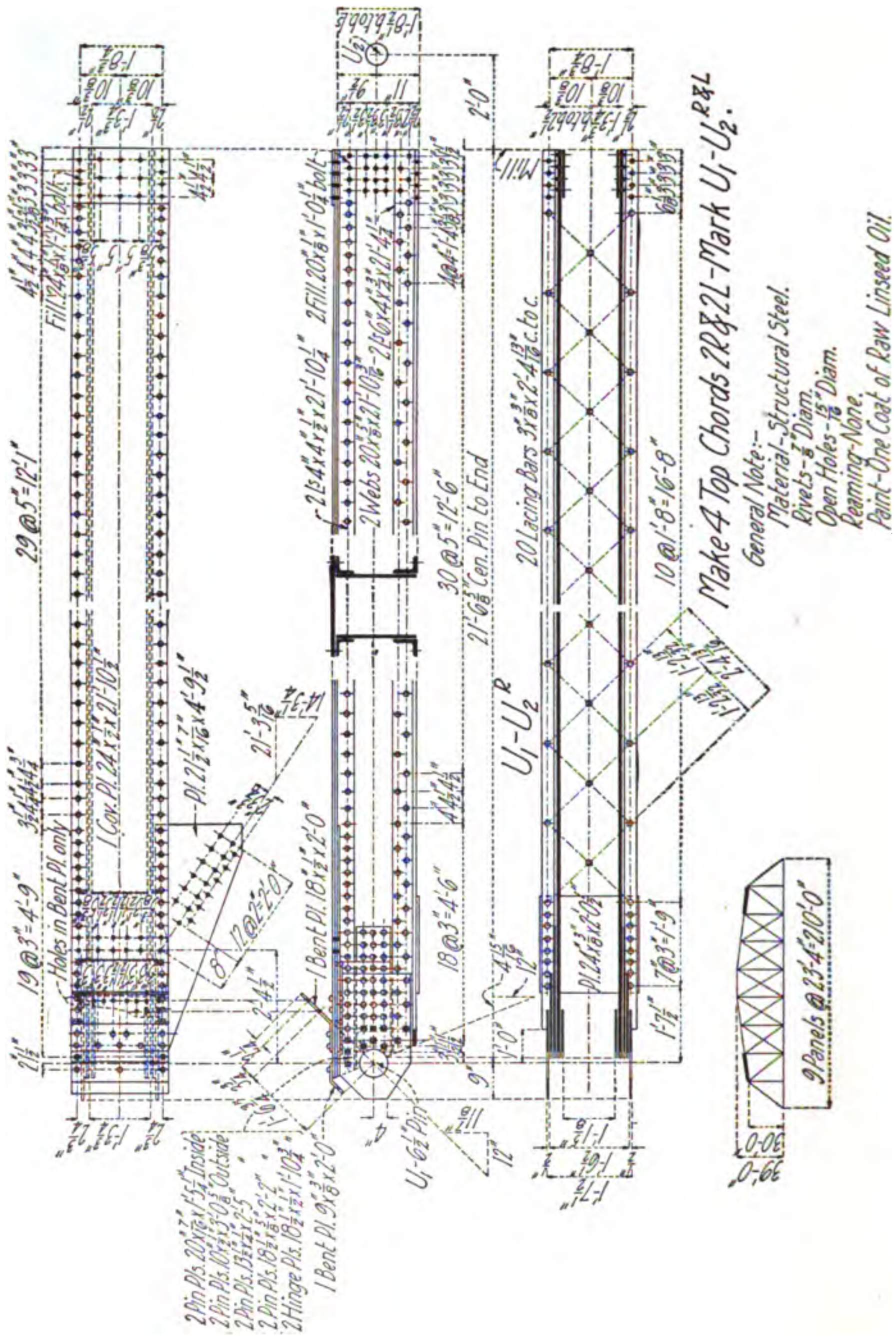
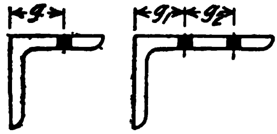



FIG. 12. SHOP DRAWING OF TOP CHORD OF PIN-CONNECTED TRUSS.

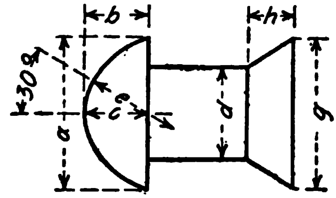



GAGES in inches

| Leg | Gage g | Max. Rivet | Leg | Gage $g_1$ | Gage $g_2$ | Max. Rivet |
|---|---|---|---|---|---|---|
| 8 | $4\frac{1}{2}$ | $\frac{7}{8}$ | 8 | 3 | 3 | $\frac{7}{8}$ |
| 7 | 4 | $\frac{7}{8}$ | 7 | $2\frac{1}{2}$ | 3 | $\frac{7}{8}$ |
| 6 | $3\frac{1}{2}$ | $\frac{7}{8}$ | 6 | $2\frac{1}{2}$ | $2\frac{1}{4}$ | $\frac{7}{8}$ |
| 5 | 3 | $\frac{7}{8}$ | 5 | 2 | $1\frac{3}{4}$ | $\frac{7}{8}$ |
| 4 | $2\frac{1}{2}$ | $\frac{7}{8}$ | When 6" L exceeds $\frac{3}{4}$" | | | |
| $3\frac{1}{2}$ | 2 | $\frac{7}{8}$ | 6 | $2\frac{1}{2}$ | 2 | $\frac{7}{8}$ |
| 3 | $1\frac{3}{4}$ | $\frac{7}{8}$ | | | | |
| $2\frac{3}{4}$ | $1\frac{5}{8}$ | $\frac{3}{4}$ | | | | |
| $2\frac{1}{2}$ | $1\frac{3}{8}$ | $\frac{3}{4}$ | | | | |
| $2\frac{1}{4}$ | $1\frac{1}{4}$ | $\frac{5}{8}$ | | | | |
| 2 | $1\frac{1}{8}$ | $\frac{5}{8}$ | | | | |
| $1\frac{3}{4}$ | 1 | $\frac{1}{2}$ | | | | |
| $1\frac{1}{2}$ | $\frac{7}{8}$ | $\frac{3}{8}$ | | | | |
| $1\frac{3}{8}$ | $\frac{7}{8}$ | $\frac{3}{8}$ | | | | |
| $1\frac{1}{4}$ | $\frac{3}{4}$ | $\frac{3}{8}$ | | | | |
| 1 | $\frac{5}{8}$ | $\frac{1}{4}$ | | | | |

MINIMUM RIVET SPACING

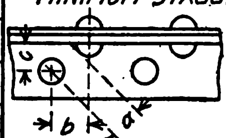
| Size of Rivet inches | Min. Distance inches |
|---|---|
| 1 | 3 |
| $\frac{7}{8}$ | $2\frac{5}{8}$ |
| $\frac{3}{4}$ | $2\frac{1}{4}$ |
| $\frac{5}{8}$ | 2 |
| $\frac{1}{2}$ | $1\frac{3}{4}$ |
| $\frac{3}{8}$ | $1\frac{1}{4}$ |
| $\frac{1}{4}$ | 1 |




PROPORTIONS OF RIVETS in inches

| Diameter of Shank d | Full Head Diameter a | Full Head Height b | Radii c | Radii e | Countersunk Diameter g | Countersunk Depth h |
|---|---|---|---|---|---|---|
| 1 | $1\frac{5}{8}$ | $\frac{11}{16}$ | $\frac{11}{16}$ | $1\frac{1}{32}$ | $1\frac{9}{16}$ | $\frac{1}{2}$ |
| $\frac{7}{8}$ | $1\frac{7}{16}$ | $\frac{39}{64}$ | $\frac{39}{64}$ | $\frac{59}{64}$ | $1\frac{3}{8}$ | $\frac{7}{16}$ |
| $\frac{3}{4}$ | $1\frac{1}{4}$ | $\frac{17}{32}$ | $\frac{17}{32}$ | $\frac{51}{64}$ | $1\frac{3}{16}$ | $\frac{3}{8}$ |
| $\frac{5}{8}$ | $1\frac{1}{16}$ | $\frac{29}{64}$ | $\frac{29}{64}$ | $\frac{43}{64}$ | 1 | $\frac{5}{16}$ |
| $\frac{1}{2}$ | $\frac{7}{8}$ | $\frac{3}{8}$ | $\frac{3}{8}$ | $\frac{9}{16}$ | $\frac{25}{32}$ | $\frac{1}{4}$ |
| $\frac{3}{8}$ | $\frac{11}{16}$ | $\frac{19}{64}$ | $\frac{19}{64}$ | $\frac{7}{16}$ | $\frac{19}{32}$ | $\frac{3}{16}$ |

MINIMUM STAGGER FOR RIVETS



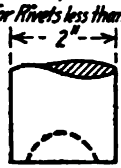

| c inches | b in inches: For $\frac{3}{4}$" Rivet $a=1\frac{1}{8}$" | b in inches: For $\frac{7}{8}$" Rivet $a=1\frac{1}{4}$" |
|---|---|---|
| $1\frac{1}{8}$ | $1\frac{1}{4}$ | $1\frac{1}{2}$ |
| $1\frac{3}{16}$ | $1\frac{3}{16}$ | $1\frac{7}{16}$ |
| $1\frac{1}{4}$ | $1\frac{1}{8}$ | $1\frac{3}{8}$ |

| c inches | b in inches: For $\frac{3}{4}$" Rivet $a=1\frac{1}{8}$" | b in inches: For $\frac{7}{8}$" Rivet $a=1\frac{1}{4}$" |
|---|---|---|
| $1\frac{5}{16}$ | $1\frac{1}{16}$ | $1\frac{5}{16}$ |
| $1\frac{3}{8}$ | $\frac{15}{16}$ | $1\frac{1}{4}$ |
| $1\frac{7}{16}$ | $\frac{7}{8}$ | $1\frac{3}{16}$ |
| $1\frac{1}{2}$ | $\frac{3}{4}$ | $1\frac{1}{8}$ |
| $1\frac{9}{16}$ | $\frac{9}{16}$ | 1 |
| $1\frac{5}{8}$ | $\frac{3}{8}$ | $\frac{15}{16}$ |
| $1\frac{11}{16}$ | 0 | $\frac{13}{16}$ |
| $1\frac{3}{4}$ | | $\frac{5}{8}$ |
| $1\frac{13}{16}$ | | $\frac{7}{16}$ |
| $1\frac{7}{8}$ | | 0 |

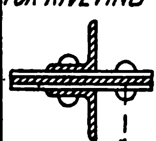
MINIMUM BUTTON SETS

For $\frac{3}{4}$" and $\frac{7}{8}$" Rivets ← $2\frac{1}{4}$" →

For Rivets less than $\frac{3}{4}$" ← 2" →



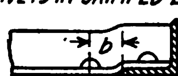
STANDARD CLEARANCE FOR RIVETING



For $\frac{7}{8}$" Rivets → $1\frac{1}{4}$" ←

For $\frac{3}{4}$" Rivets → $1\frac{1}{8}$" ←

RIVETS IN CRIMPED Ls




Distance b should $= 1\frac{1}{2}$" + 2 × thickness chord Ls, but never less than 2"

FIG. 13. AMERICAN BRIDGE COMPANY'S STANDARDS FOR RIVETS AND RIVETING.

of the column, then $W \cdot l/4 = 70I \cdot l/r \cdot y$, where $I$ = moment of inertia of the cross-section of the column $= A \cdot r^2$, where $A$ = the area of the cross-section of the column, and $y$ = the distance from the neutral axis of column to the extreme fiber in the plane parallel to the plane of the lacing bars. Then $W \cdot l/4 = 70A \cdot r^2 \cdot l/r \cdot y$, and $W = 280A \cdot r/y$. Now the shear in the column will be $W/2$, and the shear is $S = 140A \cdot r/y$, and the stress in a lacing bar will be $= 140A \cdot r \cdot \csc\theta/y$, where $\theta$ = the angle made by the bar with the axis of the column. This shows that the stresses in the lacing bars in the column with a concentric loading depend upon the make-up of the column, and are independent of the length of the column.

**Details of Compression Members.**—The details of the end-post $L_0U_1$ and top chord $U_1U_2$ of a highway camel-back truss are given in Fig. 9 and Fig. 10. "Batten plates should be placed

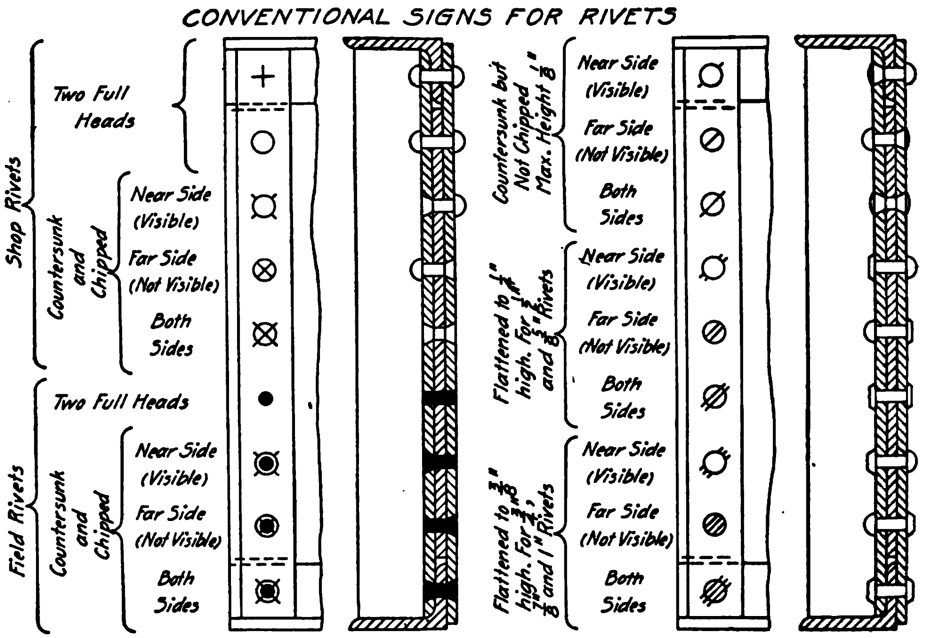



FIG. 14. CONVENTIONAL SIGNS FOR RIVETS.

as near the ends of the member as practical, should have a thickness not less than 1/40 the distance between centers of rivets at right angles to the axis of the member, and should have a length not less than the greatest width of the member or 1½ times the least width of the member." The distance between rivets is 1′ 0½″ and the thickness should be greater than ¼″ (should be $\frac{5}{16}$″), the length should not be less than 1½ times 12″ or 1′ 6″. The top cover plate satisfies the specifications for minimum thickness. The rivet spacing in the line of stress should not be greater than 16 times the thickness of the thinnest outside plate (16 × $\frac{5}{16}$″ = 5″) or 6″. This specification is not fulfilled near the center.

"The rivet spacing should not be less than three diameters of rivet." Rivets are ¾ inch and the specifications are fulfilled. "For a length from the end equal to twice the width of the member the rivet spacing should not exceed 4 times the diameter of the rivet." This specification is fulfilled for $L_0U_1$, and is practically fulfilled for $U_1U_2$. "Where pin plates are used at least one pin plate must extend 6 inches beyond the edge of the nearest batten plate." This specification is fulfilled for $U_1U_2$, but is not fulfilled for $L_0U_1$. The lacing bars satisfy the American Bridge

Company's standards for $t = c/50$, but not for $t = c/40$. The rivets in the pin plates are arranged symmetrically with reference to the pin centers. This may or may not be symmetrical with the neutral axis of the member. The details of the joint at pin $U_1$ are clearly shown. For specifications for riveting, see Appendix I. Properties of channel and plate chord sections are given in Appendix III.

Shop details of an end-post and of a top chord of a railway bridge are given in Fig. 11, and Fig. 12. Properties of built-up chord sections are given in Ketchum's Structural Engineer's Handbook.

*Angles*.—The areas of angles are given in Table 1, while the weights of angles are given in Table 2, Appendix III.

*Rivets*.—The standard form of rivets, as used by the American Bridge Company for structural and bridge work, are given in Fig. 13 together with other standards for riveting. The spacing of rivets in the legs of angles and the maximum sizes of rivets are given in Fig. 13. The American Bridge Company's conventional signs for rivets are given in Fig. 14. The spacing of rivets in the flanges of channels and I beams and the maximum sizes of rivets are given in Ketchum's Structural Engineer's Handbook.

The shearing and bearing values of rivets for several different unit stresses are given in Table 33, Appendix III.

**PINS.**—The American Bridge Company's standard bridge pins with Lomas nuts are given in Fig. 15, and in Table 24, Appendix III. Square nuts are sometimes used. The figured grip for Lomas nuts is increased as shown to make sure that the pin has a full bearing. Where square

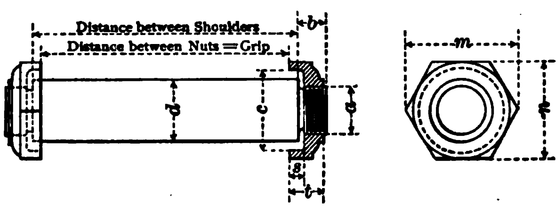



FIG. 15. BRIDGE PIN AND NUT.

nuts are used a washer should be provided at one end and the grip should be increased accordingly. In calculating the grip it is usual to assume that bars may be $\frac{1}{16}$ inch thicker than the figured thickness, that riveted members may be $\frac{1}{8}$ inch wider or narrower than the figured dimensions. Members should be packed on the pin so that the bending moments will be as small as possible. Steel pilot nuts and points for protecting the threads of the pin are shown in Fig. 17*a* and Fig. 17*b*, respectively. The allowable bending moments on pins for different fiber stresses are given in Table 27, Appendix III. The method of calculating the stresses in pins is described in detail in Chapter VI.

**LATERAL PINS.**—The American Bridge Company's standard cotter pins are given in Fig. 16, and in Table 25, Appendix III. These pins are used only for laterals and other similar members.

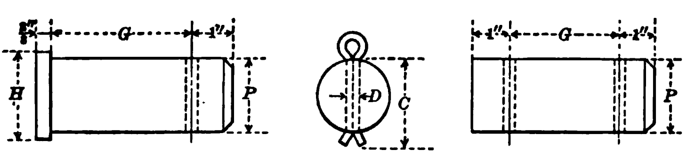
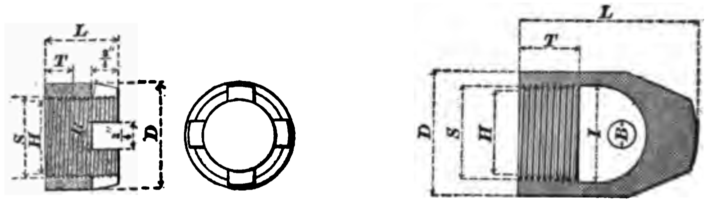



FIG. 16. COTTER PINS.

**LATERAL CONNECTIONS.**—Details of lateral connections for highway bridges are given in Tables 39 to 41 in Appendix III.



(*a*) PILOT NUT. (*b*) PILOT POINT.

FIG. 17. AMERICAN BRDIGE COMPANY PILOT NUTS AND POINTS.

**SHOES AND PEDESTALS.**—The bridge rests on shoes or pedestals, the loads being transferred to the shoes in pin-connected bridges by means of pins, and through pins or through the

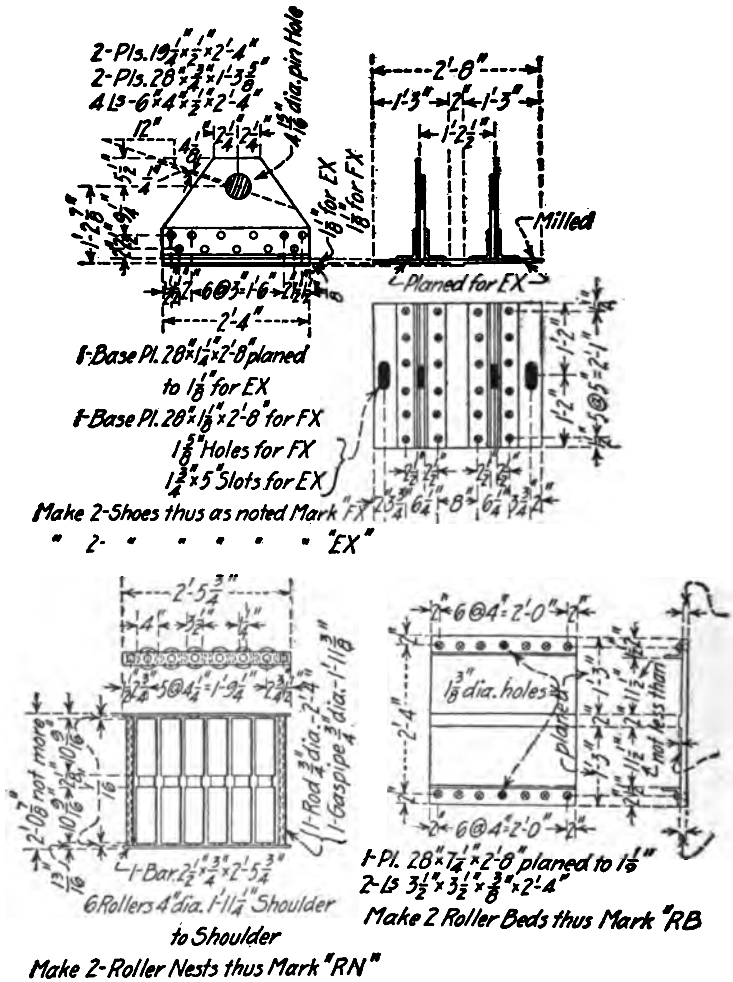



FIG. 18. DETAILS OF SHOES AND ROLLER NEST FOR A HIGHWAY BRIDGE.

riveted joints in riveted bridges. The shoes at the expansion ends of the bridge are placed on smooth sliding plates for bridges of less than 60 to 80 ft. span, and on nests of rollers or rockers for spans of greater length. The action of the rollers under the expansion ends of riveted bridges

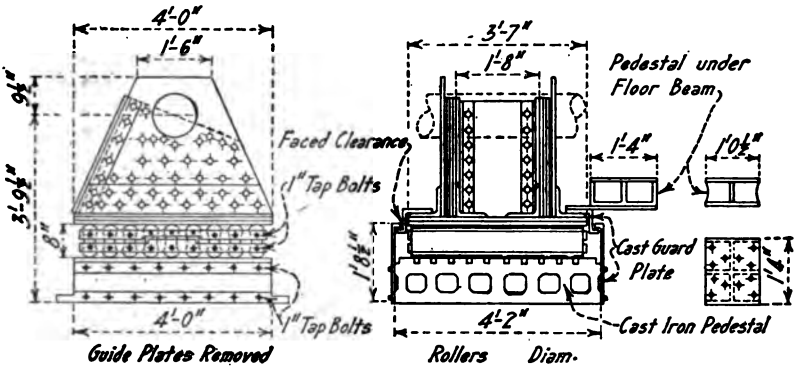



FIG. 19. DETAILS OF A STEEL SHOE WITH SEGMENTAL ROLLERS.

will be much more satisfactory if the shoes are pin-connected to the truss the same as for pin-connected trusses. Rollers should be made with as large diameters as practicable in order to reduce the pressure on the base plate and also to reduce the resistance to movement. Experience

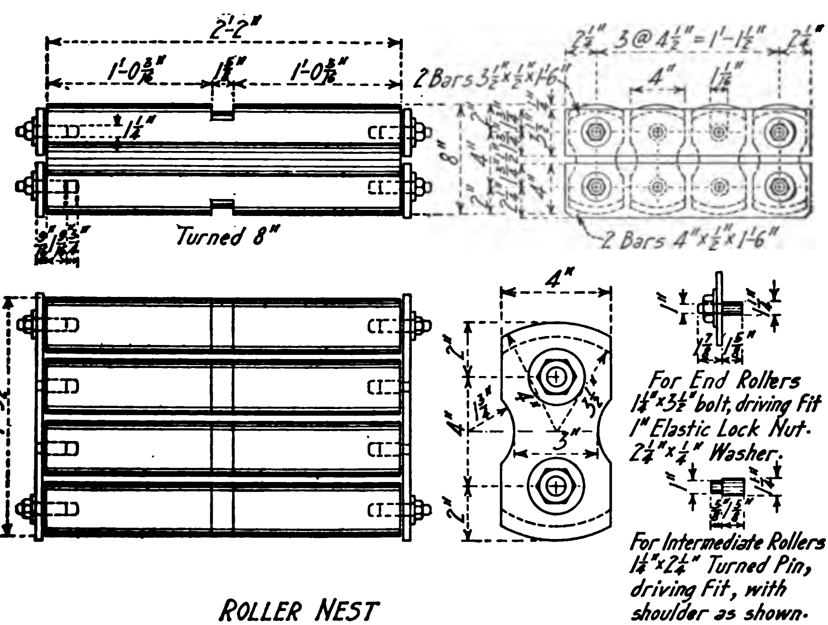



FIG. 20. DETAILS OF SEGMENTAL ROLLERS.

shows that even for light bridges rollers smaller than 3 in. diameter are practically worthless. Details of a roller bearing for a highway bridge are shown in Fig. 18. Additional details are shown in Chapters XIII and XIV. The bed plate which supports the rollers should be made of

sufficient thickness to give a rigid bearing for the rollers. To economize space, segmental rollers, as shown in Fig. 19, are often used for heavy spans. Details of segmental rollers are shown in Fig. 20. The allowable pressure of steel rollers on a steel bed plate is given in the specifications in Appendix I as 600 $d$ pounds per lineal inch of roller, where $d$ is the diameter of the roller. Segmental rollers permit smaller and thinner bed plates than circular rollers due to the closer spacing of the rollers.

The Illinois, Iowa, Wisconsin and Minnesota Highway Commissions use cast iron rockers on steel bed plates in place of steel rollers for the expansion ends of highway bridges. Details of the standard rocker used by the Iowa Highway Commission are shown in Fig. 5, Chap. XIV. This rocker is used for spans from 70 ft. to 150 ft. The bed plate is increased in size for spans of over 120 ft. Details of rockers are shown in Fig. 5, Fig. 6, and Fig. 15, Chapter XIII, and in Fig. 3, Fig. 4, Fig. 5 and Fig. 17, Chapter XIV. The standard rocker of the Illinois Highway Commission is practically identical with the Iowa Highway Commission standard. The standard rocker of the Wisconsin Highway Commission is somewhat heavier, as can be seen by a comparison of the details. Cast iron rockers are commonly designed for an allowable load per lineal inch of rocker of 300 $d$, where $d$ is the diameter of the rocker. In specifications in Appendix I, cast iron rockers are to be designed for an allowable load per lineal inch of 300 $d$, a cross-bending stress of 3,000 lb per sq. in., and a shearing stress of 1,500 lb. per sq. in. Calculations made by the author on the strength of the Iowa standard rocker for a span of 70 ft., gave a maximum cross-bending stress of 4,500 lb. per sq. in. at the center, and a shear of 1,500 lb. per sq. in. The stresses will be greater when the rocker is used for longer spans. The Wisconsin rocker shown in Fig. 4, Chapter XIV, is somewhat more rigid than the Iowa standard. The author has modified the Iowa standard rocker in the designs in Fig. 16, Chapter XIII, and Fig. 18, Chapter XIV, by increasing the thickness of the rocker at the center from 2½ in. to 3 in.

It is usual to specify that a movement produced by a variation of 150 degrees Fahr. be provided for. The coefficient of expansion of steel is approximately 0.000,006,7 per degree Fahr., which makes it necessary to provide for approximately one inch of movement for each 80 ft. of bridge span.

Where both bridge seats are of the same height, the fixed end is carried on cast iron pedestal blocks. The blocks are usually made with recesses (honey-combed) to reduce the weight.

**FENCE AND HUB GUARDS.**—The fence on steel bridges is commonly made of two lines of channels or two lines of angles with angle posts. Posts should not be spaced farther apart than 8 ft. to 10 ft.

The simplest form of fence is that shown in Fig. 22. The posts are made of 4 in. × 4 in. pieces and are spaced about 8 ft. apart. The top railing is made of two pieces 2 in. × 4 in., while the side piece is a 2 in. × 8 in. Similar details are used for steel joists.

The 4 in. × 6 in. felloe guard should be firmly bolted to the floor. Blocks of wood 1 or 2 inches thick called "shims" are sometimes placed between the felloe guard and the floor. Shims are of questionable utility, and should not be used. A gas pipe fence with angle rail is shown in Fig. 21. A gas pipe railing with gas pipe posts is shown in Fig. 23. The posts should be spaced not more than 8 ft. apart. The rail in Fig. 23 was used in the Pennsylvania Ave. Subway, Philadelphia, and was furnished at $0.95 per lineal foot. An ornamental fence with pipe top rail and cast iron newel posts is shown in Fig. 24. The rail in Fig. 24 was used on the same contract as that shown in Fig. 23, and was furnished at $2.00 per lineal foot. Details of the fence and light poles for the 20th St. Viaduct, and the fence on 23d St. Viaduct, Denver, Colo., designed by Mr. H. S. Crocker, consulting engineer, are shown in Fig. 25.

**WATERPROOFING.**—Concrete mixed so as to give maximum density and reinforced with not less than one-third of one per cent of reinforcement to prevent cracks is quite impervious. The water-tightness of the concrete may be increased by plastering the top of the slab with 1–2 Portland cement mortar.

Excellent results will be obtained by painting the side of the concrete exposed to hydrostatic pressure with a coal tar paint, made by mixing 16 parts of coal tar with 4 parts of Portland cement and 3 parts of kerosene. The Portland cement should first be stirred into the kerosene,

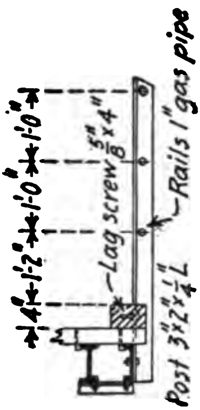


FIG. 21.

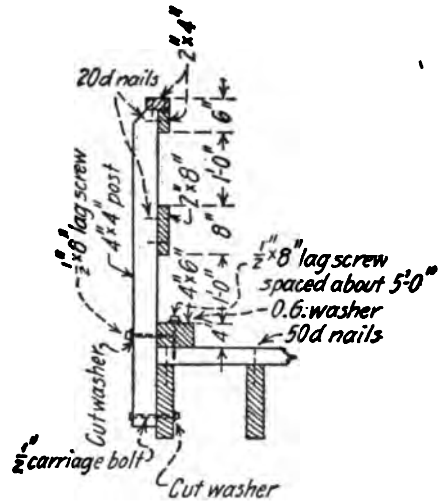


FIG. 22.

forming a creamy mixture, the mixture is then stirred into the coal tar. In cold weather the paint should be warmed. The paint sinks into the concrete an ⅛ in. or so, and sticks well to the surface. Not less than two coats should be applied. The author has obtained excellent results with coal tar paint in waterproofing the backs of retaining walls and bridge abutments.

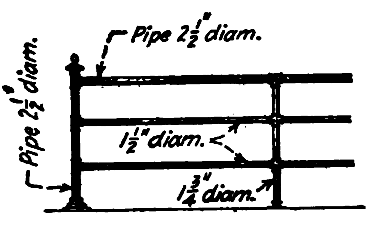


FIG. 23.

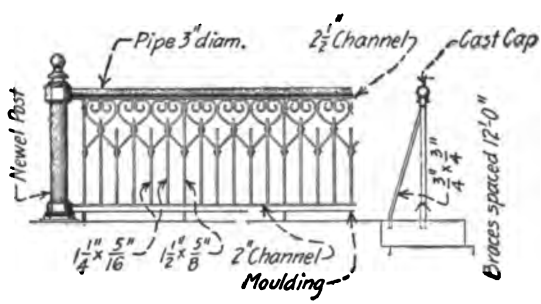


FIG. 24.

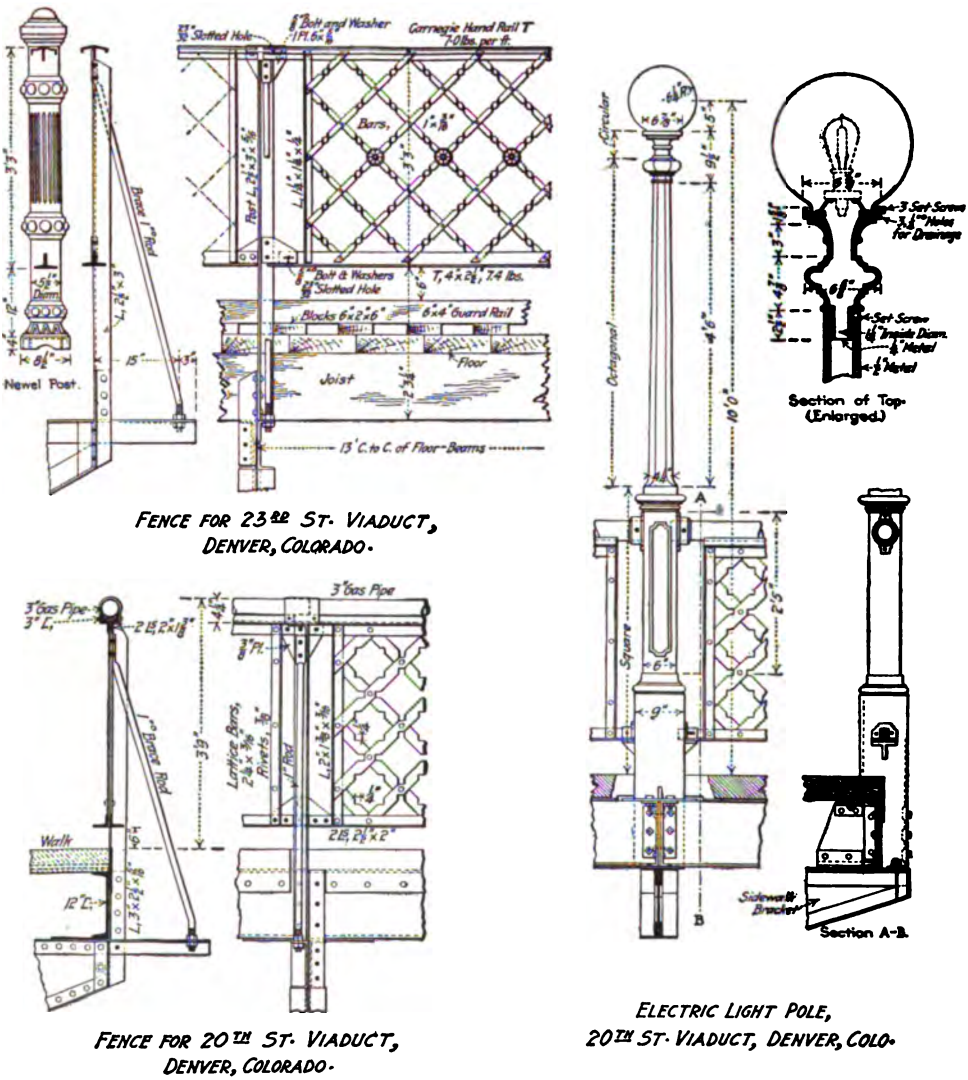
Asphalt and coal tar are also used for waterproofing concrete structures.

The most effective waterproofing is to apply several layers of burlap soaked in asphalt or tar. Each layer is laid shingle fashion and each layer is mopped with tar or asphalt before the next layer is applied. For detail specifications for waterproofing bridge floors by this method, see the author's "Structural Engineer's Handbook."

**PROTECTION OF OVERHEAD BRIDGES.**—Where bridges span railroad tracks the steel in the floor system, lower chords and other exposed parts is exposed to the sand blast action of

the locomotive exhaust and the corrosive effect of the locomotive gases. The clearance above the locomotives is usually small and the steel work must be given special protection.

The following methods have been used for protecting the steel work in overhead bridges.




FENCE FOR 23RD ST. VIADUCT, DENVER, COLORADO.

FENCE FOR 20TH ST. VIADUCT, DENVER, COLORADO.

ELECTRIC LIGHT POLE, 20TH ST. VIADUCT, DENVER, COLO.

FIG. 25. STEEL FENCE FOR HIGHWAY BRIDGES.

(*a*) *Cast Iron Plates.*—Cast iron plates have been used on several railroads. On the Cleveland Short Line Nickel Plate Grade Crossing, Cleveland, Ohio, the cast iron ceiling plates were 2 ft. 4½ in. wide and 4 ft. long, and about ⅝ in. thick with 3 in. ribs. This method is quite satisfactory but expensive.

(*b*) *Asbestos Sheathing.*—Asbestos sheathing in sheets 4 ft. by 4 ft. by ⅜ in. thick have been extensively used for protecting overhead bridges. The sheets are fastened to light angle members. The bolts extending below the asbestos sheets are covered with a mixture of asbestos fiber and Portland cement, mixed 1 to 3. It is claimed that an asbestos protection 2 ft. on each side of the center line of the track gives a satisfactory protection. The use of asbestos sheathing is quite satisfactory.

(*c*) *Wood Boards.*—Boards swell and shrink and give an inadequate protection. There is also a fire hazard.

(*d*) *Paint and Cement Coating.*—A coat of red lead and linseed oil is applied, mixed in the proportions of 25 lb. of red lead per gallon of oil. While the paint is still wet, clean, coarse sand is thrown against it so that about one-half the diameter of the grains is embedded in the paint. After the paint is dry two coats of cement grout, creamy in consistency and mixed in the proportions of one sand to one Portland cement, is applied with brushes. This method is relatively inexpensive and gives fair protection, where there is adequate clearance.

(*e*) *Concrete Coating With Cement Gun.*—The metal to be protected is covered with wire mesh or expanded metal reinforcement. The cement mortar is applied to the reinforcement by means of a cement gun. The cement mortar should be not less than 1½ in. thick. The results obtained by this method are satisfactory where conditions are not too severe. This method has the added advantage that the entire structure may be coated and the steel part of the structure can be made to harmonize with the concrete. Fairly satisfactory results have been obtained by applying the mortar with trowels. For description of the cement gun and of its work, see Journal Western Society of Engineers, Vol. 19, 1914, pp. 272–318.

**PAINTING.**—Paint is a combination of a pigment and a vehicle. The common pigments used for painting structural steel are red lead, white lead, iron oxide, zinc, graphite or carbon. Linseed oil is the best and most common vehicle. Linseed oil oxidizes, and the hardening of the film can be hastened by the addition of Japan drier, turpentine or other drier. The drier weakens the film and should not be used. By heating raw linseed oil for the required time the film hardens much more rapidly. Boiled linseed oil should be used for structural steel paints. The pigment should be thoroughly ground into and be mixed with the oil. A common rule for mixing paint is to mix with each gallon of linseed oil a weight of pigment in pounds equal to three to four times the specific gravity of the pigment. This rule gives the following weights per gallon of linseed oil: red lead, 25 to 33 lb.; white lead, 19 to 26 lb.; zinc, 15 to 21 lb.; iron oxide, 15 to 20 lb.; lampblack, 8 to 10 lb.; graphite, 8 to 10 lb.

The covering capacity of a paint depends upon the uniformity and thickness of the coating. To obtain any given thickness of paint requires practically the same amount of paint, whatever its pigment may be. A graphite or carbon paint will cover nearly twice as much surface as a good red lead paint. Red lead or iron oxide will cover a surface of 500 to 600 sq. ft. with one coat; while carbon or graphite will cover a surface of from 750 to 800 sq. ft. Light structural work will average about 250 sq. ft., and heavy structural work about 150 sq. ft. per net ton of metal. It is the common practice to estimate ½ gallon of paint per ton for the first coat, and ⅜ gallon of paint for the second coat per ton of structural steel for average conditions.

The paint should be thoroughly brushed out with a round brush to remove all the air. The paint should be mixed only as wanted, and should be kept well stirred. When it is necessary to apply paint in cold weather, it should be heated to a temperature of 130 to 150 degrees F.; paint should not be put on in freezing weather. Paint should not be applied when the surface is damp, or during foggy weather. The first coat should be allowed to stand for three or four days, or until thoroughly dry, before applying the second coat. If the second coat is applied before the first coat has dried, the drying of the first coat will be very much retarded.

Before applying the paint the surface should be thoroughly cleaned of all dirt, grease, scale, rust or dead paint. The metal may be cleaned by scraping and brushing with wire brushes, or by means of a sand blast. The cost of cleaning steel with a sand blast will vary from $1.50 to

$2.00 per ton, not including rent for the apparatus. The paint should be applied immediately after cleaning, and before rusting has started.

For additional data on paints see the author's "Structural Engineer's Handbook."

The most common shop coat is red lead paint, although some engineers prefer linseed oil alone, or mixed with sufficient lampblack to give a black coating.

The specifications for paint and painting as required by various commissions follow. The author's specifications for field painting are given in Appendix I.

**Examples of Paint Specifications.**—The practice in painting highway bridges will be shown by the following abstracts from specifications.

**Iowa Highway Commission.—Painting Metal Structures.**—One shop coat and one field coat are required. All metal must be cleaned of rust, scale, dirt or grease and must be dry before applying shop coat. Shop coat to be applied after assembling and riveting. Parts not accessible after erection to be painted two coats. Machined surfaces to be coated with lead and tallow. Shop paint shall be one of the following pigments mixed with pure linseed oil or China wood oil, with not more than 10 per cent Japan drier. Red lead paint, not less than 65 per cent pigment; sublimed blue lead paint, not less than 60 per cent pigment; sublimed lead sulfate, not less than 60 per cent pigment; basic lead or zinc chromate paint, not less than 60 per cent pigment; iron oxide paint, not less than 55 per cent pigment; graphite paint, not less than 35 per cent pigment. Iron oxide is to be mixed with not less than 10 per cent basic lead or zinc chromate. Natural graphite is to be mixed with not less than 20 per cent basic lead or zinc chromate. All percentages are in terms of weight of finished paint.

After structure is erected complete it is to be cleaned of dirt, grease or oil and is to be given one coat of paint. Field paint shall be one of the following pigments mixed with pure boiled linseed oil or China wood oil, with not more than 10 per cent Japan drier. Red lead, not less than 55 per cent pigment; sublimed blue lead, not less than 50 per cent pigment; sublimed sulfate of lead, not less than 55 per cent pigment; iron oxide, not less than 50 per cent pigment; pure graphite (natural), or pure carbon, not less than 25 per cent pigment. Red lead is to have some tinting pigment in sufficient quantities to eliminate fading of straight red lead paint.

No painting shall be done in wet weather, or when the temperature is not above 45 degrees F for at least 10 hours per day.

Before repainting old bridges all loose paint, scale, rust and dirt shall be removed by a sand blast, with metal scrapers, wire brushes, or the painters torch. The quality of cleaning shall be equal to that produced by the sand blast. First coat of paint shall be applied as soon as practicable after cleaning. If rusting results before painting, surface must be recleaned. One coat of prime paint and one coat of field paint shall be applied. The first coat shall have time to dry before applying the second coat.

Wood structures shall be given two coats of white paint made by mixing pure white lead 65 per cent, pure zinc white 20 per cent, and not more than 15 per cent inert material. The paint shall contain from 60 to 65 per cent pigment and pure boiled linseed oil, Japan drier shall not exceed 10 per cent.

**Illinois Highway Commission.**—Three coats of paint shall be used as follows: A shop coat of pure sublimed blue lead and pure boiled linseed oil; a second coat, applied in the field of a mixture of 80 per cent pure sublimed white lead and 20 per cent pure blue lead and pure boiled linseed oil; the third coat to be of pure sublimed lead and pure linseed oil. The mixed paint shall contain not less than 50 nor more than 54 per cent of pigment by weight.

The specifications for details of applying shop and field coats, and repainting old bridges are practically the same as given in the specifications of the Iowa Highway Commission.

**Minnesota Highway Commission.**—Shop coat of red lead made by mixing 25 lb. red lead, 94 per cent pure, with one gallon pure boiled linseed oil. Field coat to be an approved graphite paint. If specified, trusses and railing and rail posts of beam spans shall be painted with pure sublimed white lead and pure boiled linseed oil so mixed that the paint contains not less than 50 nor more than 54 per cent pigment by weight. The use of tar and asphaltum paints is prohibited.

**Michigan Highway Commission.**—Shop coat to be red lead and linseed oil. Two field coats to be of very distinct colors. The final coat being black, either red lead and a linseed oil; or other paint as specified.

**Virginia State Highway Commission.**—Shop coat of red lead and oil. Two field coats, the first pure sublimed white lead and oil; the second coat of pure white lead, with four ounces of lamp black in oil and eight ounches of French ochre per 100 lb. of pigment. Mixed paint to have 55 per cent of pigment by weight, approximately 18 lb. of pigment per gallon of linseed oil.

**Oregon Highway Commission.**—Shop coat of best carbon primer or red lead and linseed oil in proportion of 30 lb. paste to one gallon of oil. Two field coats as specified. All three coats of paint to be of different color.

**Pennsylvania State Highway Department**—Shop coat to be red lead (85 per cent pure) mixed with pure raw linseed oil. Field coats to be one coat of red lead (85 per cent pure), and two coats of white lead and zinc white, mixed in proportion of 75 lb. white lead to 25 lb. zinc white, in pure linseed oil without turpentine or drier. The final coat of paint may be tinted as desired.

**Engineering Institute of Canada.**—Shop coat of pure red lead and lampblack mixed in the proportions of 25 lb. of red lead, 4 oz. of lampblack, and one gallon boiled linseed oil. Shall not be thinned with turpentine, benzine or other liquids, and no drier will be allowed. Parts inaccessible after erection to be given two coats of shop paint. Planed and turned surfaces to be coated with white lead mixed with tallow before leaving the shop. Metal surfaces thoroughly cleaned before applying shop coat, using sand blast if necessary and dusted with stiff bristle brush. Two field coats of approved paint after erection. Field coats to be of different colors.

In "Bridge Engineering" Mr. J. A. L. Waddell after a thorough discussion recommends a shop coat of red lead ground in linseed oil, and two field coats of carbon or graphite paint.

**Painting Railroad Bridges.**—An analysis of the practice of painting railway bridges (Proc. A. R. E. A., 1915) shows:

(1) For a shop coat, 29 out of 50 used red lead; 5 out of 50 used linseed oil alone; 6 out of 50 used linseed oil, painting parts in contact with paint; 8 out of 50 used graphite and carbon pigments, including lampblack; 2 out of 50 used miscellaneous paints.

(2) For field coats on new steel, 12 out of 48 used red lead; 24 out of 48 used carbon or graphite or both; 12 out of 48 used miscellaneous paints.

(3) For maintenance, 13 out of 46 used carbon; 7 out of 46 used graphite; 4 out of 46 used both carbon and graphite; 11 out of 46 used red lead; 11 out of 46 used miscellaneous paints.

**Painting Timber Structures.**—Ohio State Highway Department specifies three coats of white paint for timber structures. The white paint is made by mixing 75 lb. white lead in oil, 25 lb. zinc oxide in oil, 4 gallons raw linseed oil, and 1 quart drier.

For specifications for painting timber structures, see § 49, General Specifications for Timber Bridges and Trestles in Chap. XVI.

# CHAPTER XVI.

## DESIGN OF TIMBER BRIDGES AND TRESTLES.

**Introduction.**—Timber highway bridges were formerly quite generally used, and are still in use for temporary structures and in localities where transportation is difficult and suitable timber is available. Timber truss bridges are usually made with the Howe type of truss with timber top and bottom chords and diagonal braces, and with steel rods for vertical ties. Timber floorbeams and joists, and a timber flooring are used. Timber Howe through truss bridges of more than about 70 ft. span are commonly housed to protect the bridge timbers from decay.

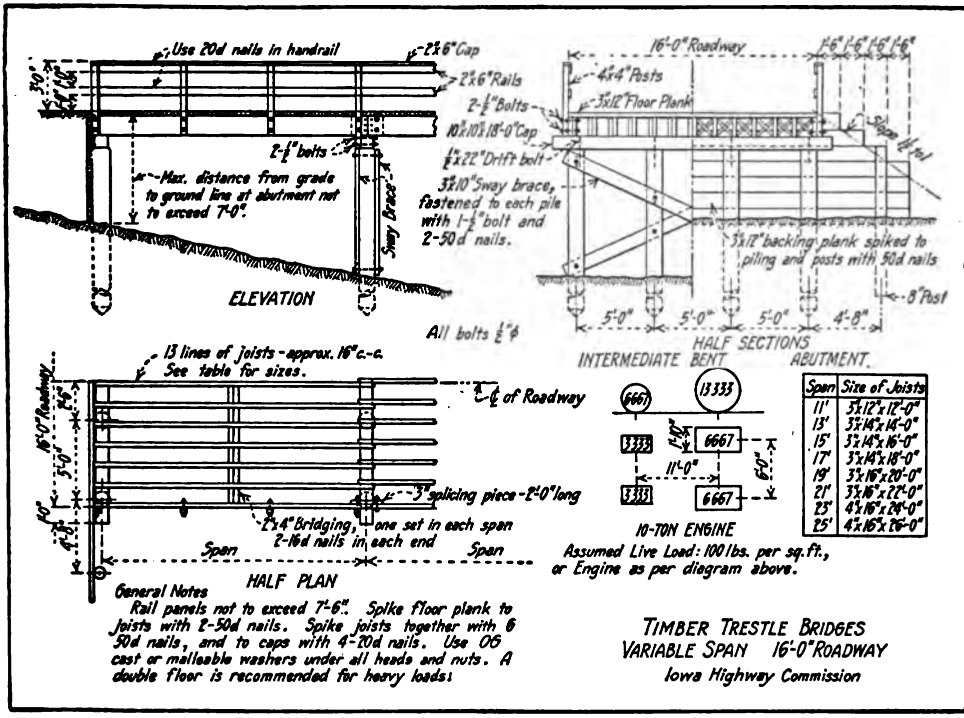



FIG. 1. TIMBER TRESTLE BRIDGE. IOWA HIGHWAY COMMISSION.

Timber Howe through truss bridges are commonly made with cast iron packing blocks to increase the resistance to crushing, for the reason that the bearing on inclined surfaces of timber is much less than square end bearing. Low truss timber bridges are commonly made without cast iron details.

In the west, combination timber and steel highway bridges have been quite generally used, and when well designed and constructed give excellent service. Combination bridges are usually made with the Pratt type of truss, with timber top chords, posts, struts, floorbeams, joist, and floor,

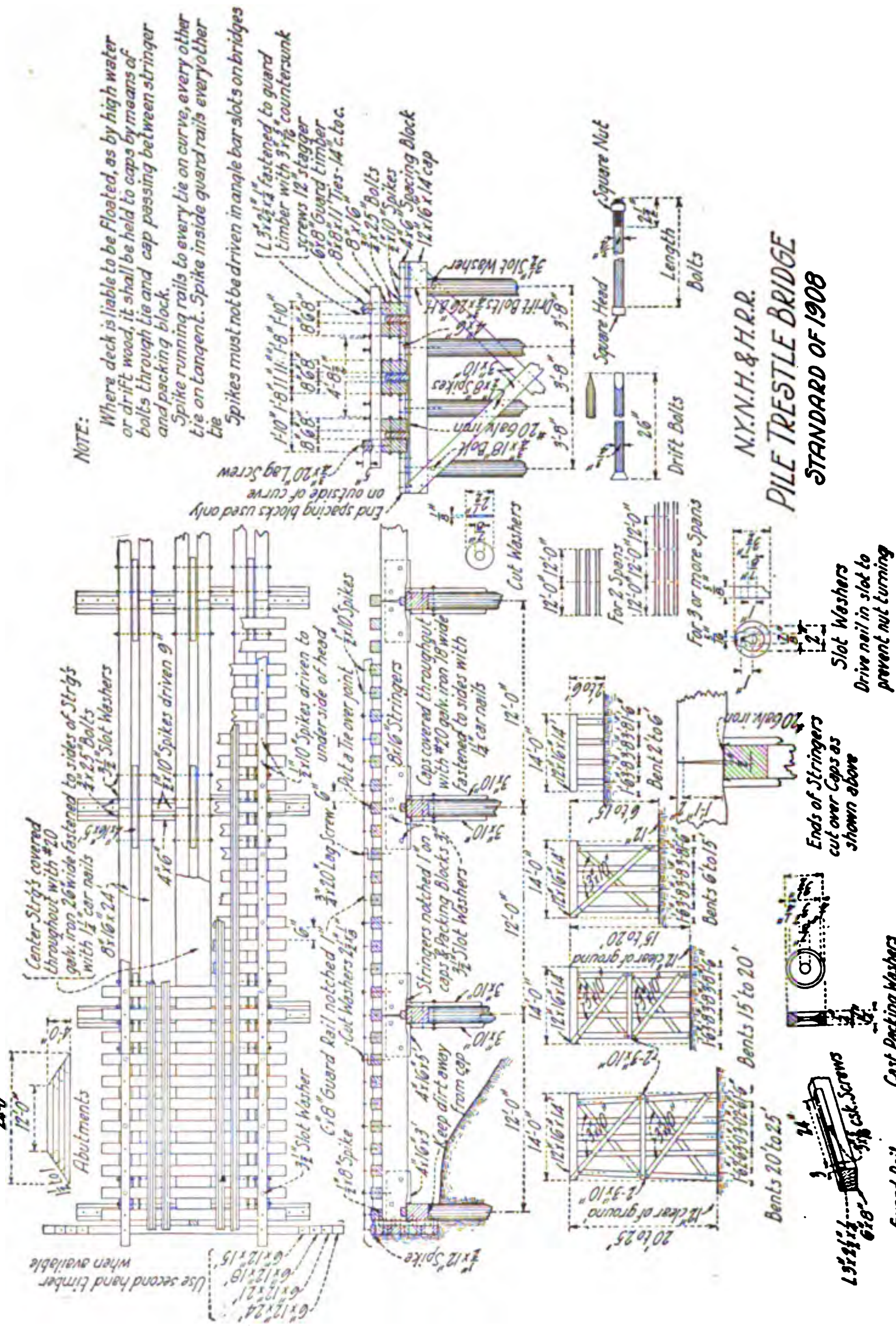



FIG. 2. PLANS OF A PILE TRESTLE.

and with steel lower chords, diagonals and lateral rods. Howe trusses are occasionally made with steel angle lower chords; or with steel bars fastened to the timber lower chords to take the tension while the timber gives the necessary stiffness.

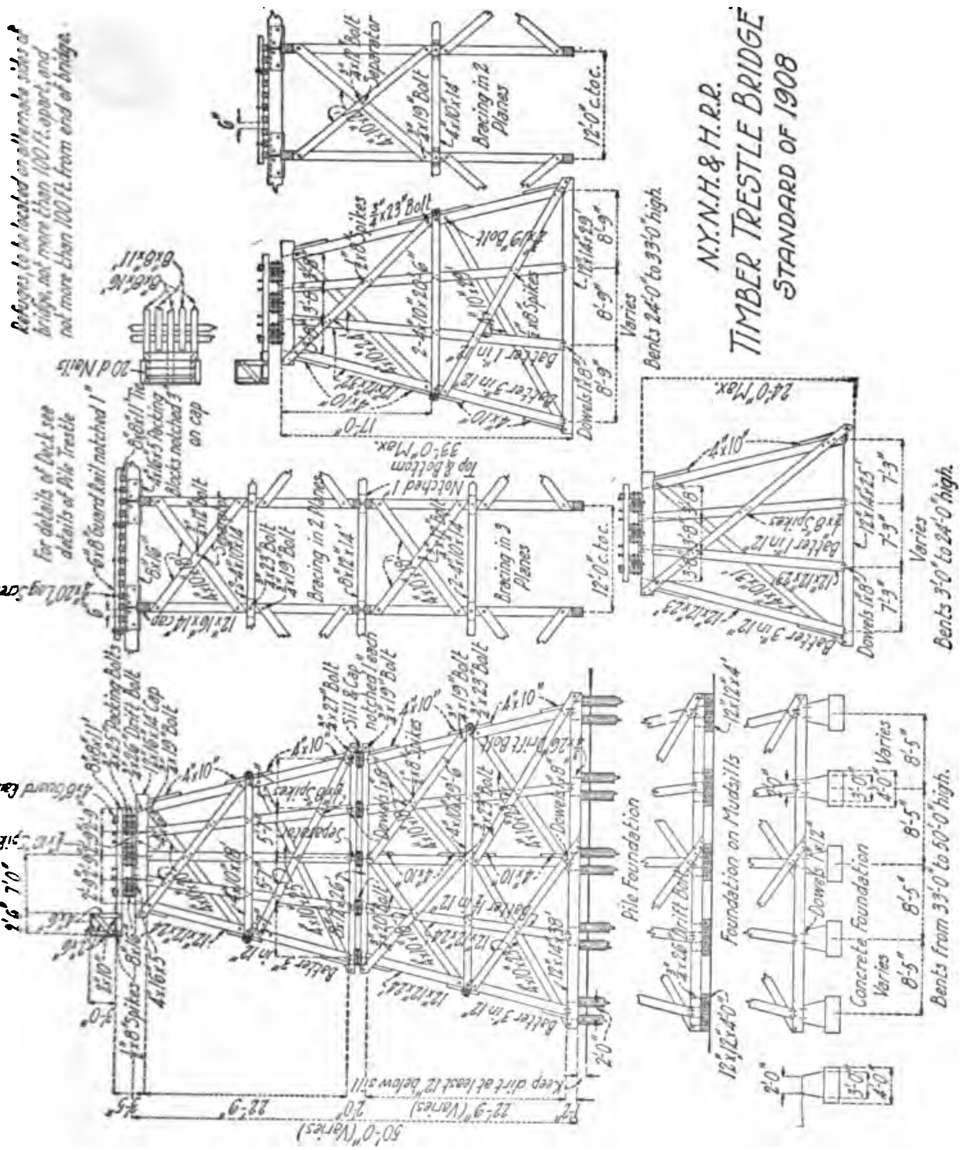



FIG. 3. PLANS OF A FRAME TRESTLE.

**TIMBER TRESTLES.**—Timber trestles may be made by using pile bents or by using framed bents. The framed bents have the lower sills resting on piles or on concrete or stone blocks. Timber mudsills may be used in temporary structures, but should never be used in permanent construction.

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Details of a timber highway trestle bridge as designed by the Iowa Highway Commission are shown in Fig. 1. The allowable stresses were the same as for timber bridges. The specifications of the Iowa Highway Commission provide for creosoting the timbers and piles in permanent timber pile trestle bridges.

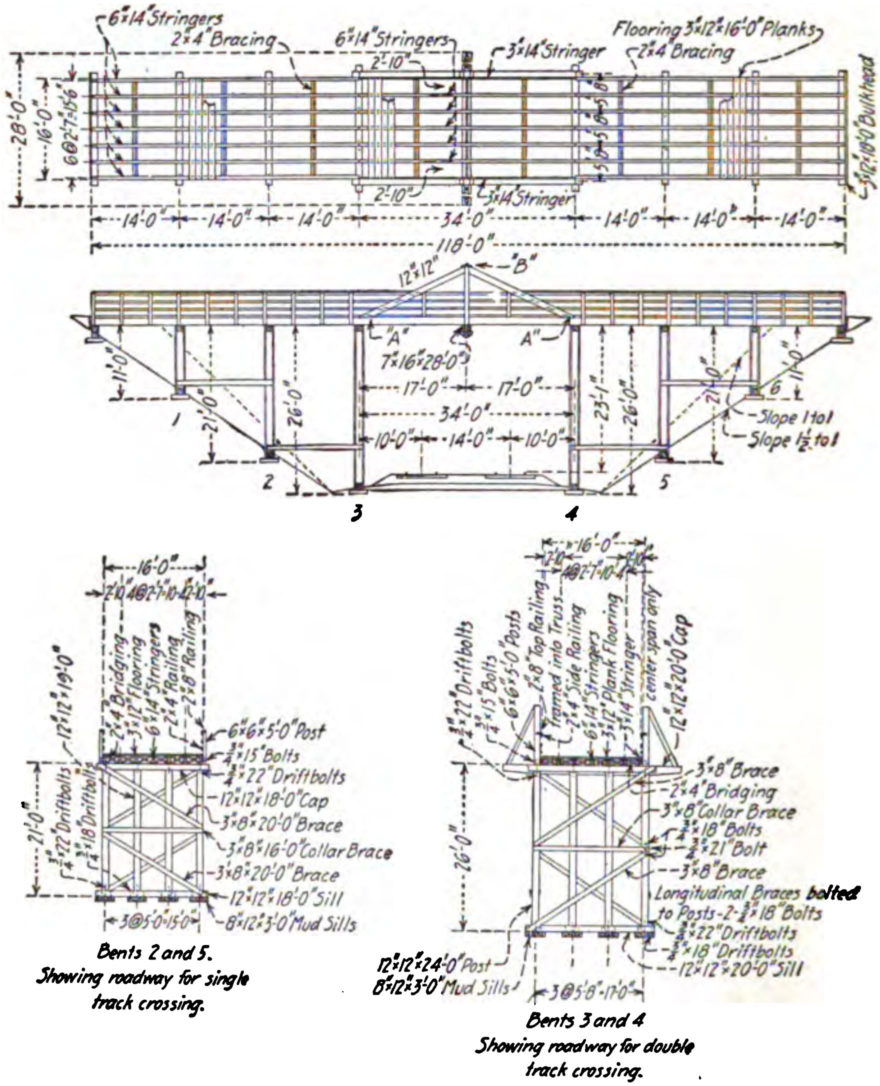



FIG. 4. HIGHWAY CROSSING. ILLINOIS CENTRAL RAILROAD.

Details of a pile trestle for a railroad are given in Fig. 2, and of a frame trestle for a railroad are given in Fig. 3. These plans represent standard practice in the design of timber trestle bridges for railroads, and furnish excellent details for the engineer who is designing highway bridge trestles.

**TIMBER TRUSS BRIDGES.**—A timber highway bridge designed by the Illinois Central R. R. for an overhead crossing is shown in Fig. 4. Details of the joints are shown in Fig. 5.

The details of a standard timber truss bridge as designed by the Iowa Highway Commission are given in Fig. 6. Standard plans have been prepared for two and three panel timber bridges with panel lengths of 15 ft., 17 ft. and 19 ft. The two panel bridge with 30 ft. span has the same

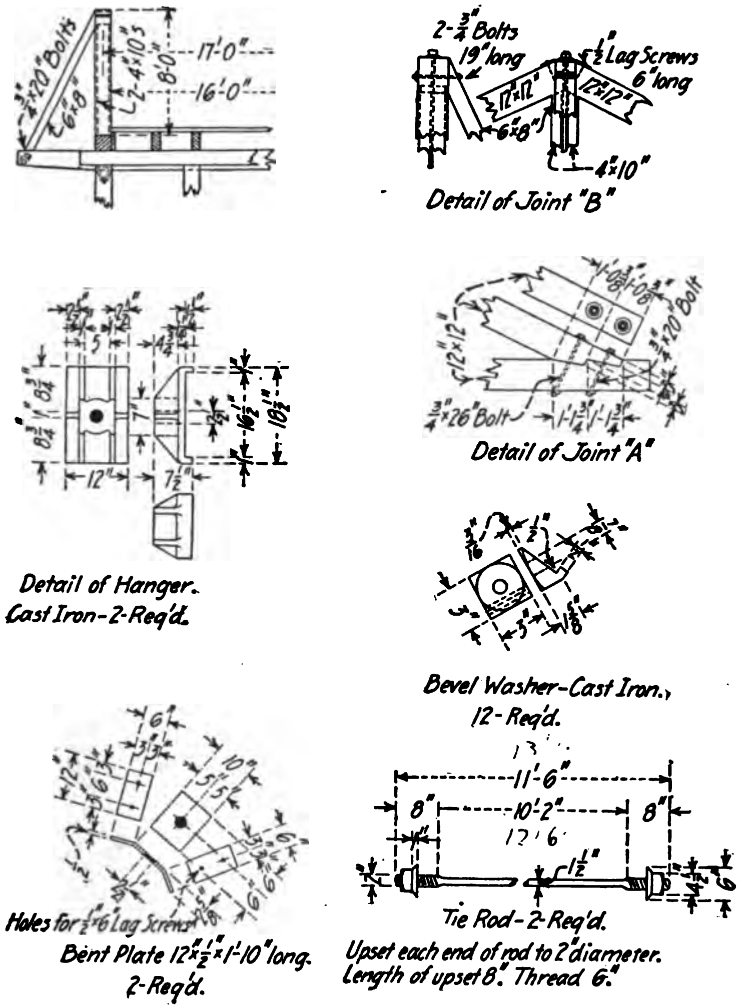
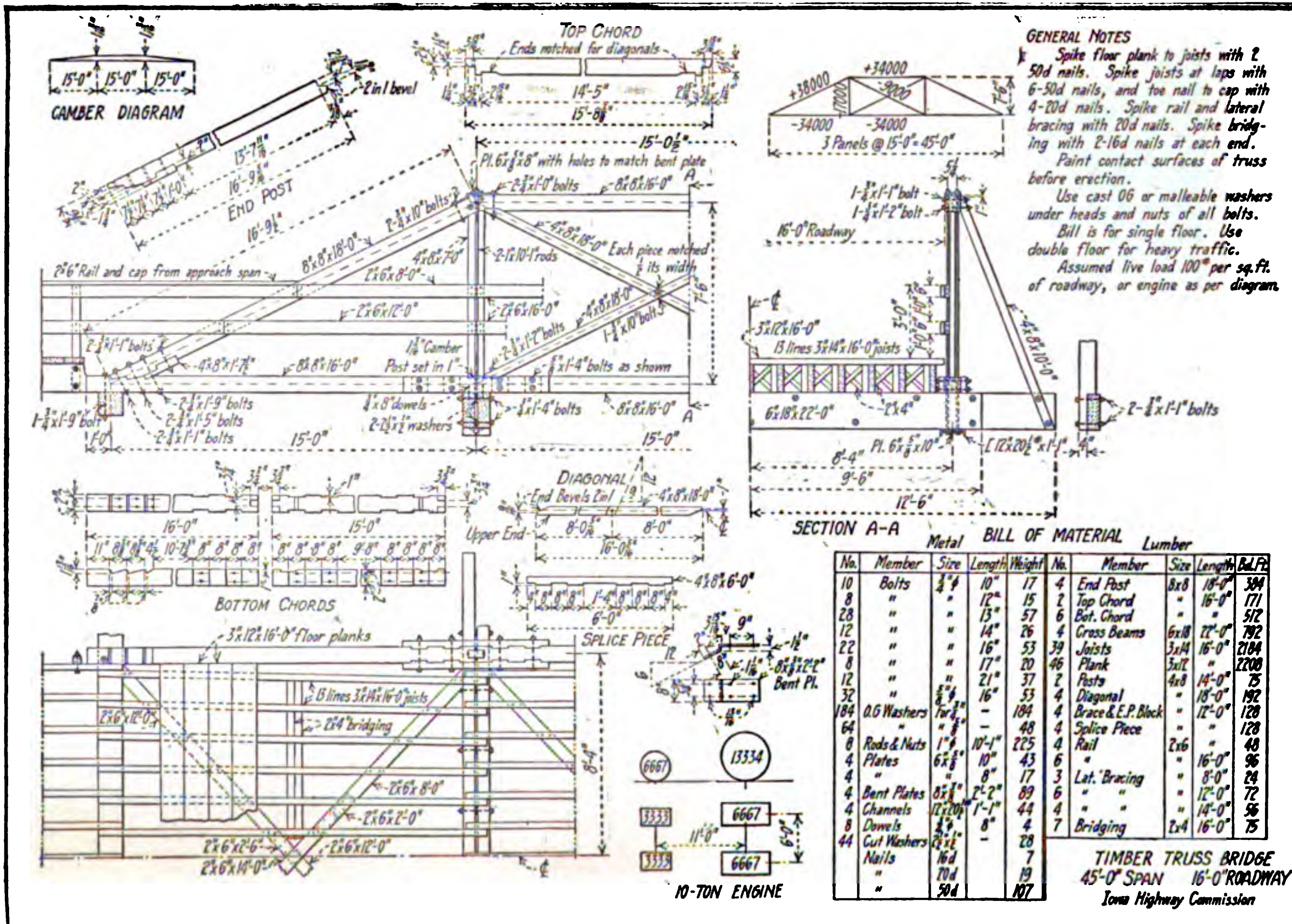



FIG. 5. DETAILS OF HIGHWAY CROSSING. ILLINOIS CENTRAL RAILROAD.

sections and the same details as the end panels of the 45 ft. span shown in Fig. 6, except the extra block at the lower end of the end-post is omitted.

The Iowa Highway Commission uses the following allowable stresses in the design of timber structures in lb. per sq. in. Douglas fir, white oak, and long leaf yellow pine. Bending on extreme fiber, 1,500; tension with grain, 1,500; compression with grain, 1,500; shear parallel to grain, 150; longitudinal shear in beams, 110; compression perpendicular to grain, 300. Stresses in columns,




BILL OF MATERIAL

| Metal No. | Member | Size | Length | Weight | Lumber No. | Member | Size | Length | Bd. Ft. |
|---|---|---|---|---|---|---|---|---|---|
| 10 | Bolts | ¾"φ | 10" | 17 | 4 | End Post | 8x8 | 18'-0" | 384 |
| 8 | " | " | 12" | 15 | 2 | Top Chord | " | 16'-0" | 171 |
| 28 | " | " | 13" | 57 | 6 | Bot. Chord | " | " | 512 |
| 12 | " | " | 14" | 26 | 4 | Cross Beams | 6x18 | 22'-0" | 792 |
| 22 | " | " | 16" | 53 | 39 | Joists | 3x14 | 16'-0" | 2184 |
| 8 | " | " | 17" | 20 | 46 | Plank | 3x12 | " | 2208 |
| 12 | " | " | 21" | 37 | 2 | Posts | 4x8 | 14'-0" | 75 |
| 32 | " | ⅝"φ | 16" | 53 | 4 | Diagonal | " | 18'-0" | 192 |
| 184 | O.G Washers | For ¾" | – | 184 | 4 | Brace & E.P. Block | " | 12'-0" | 128 |
| 64 | " | " ⅝" | – | 48 | 4 | Splice Piece | " | " | 128 |
| 8 | Rods & Nuts | 1"φ | 10'-1" | 225 | 4 | Rail | 2x6 | " | 48 |
| 4 | Plates | 6x⅝" | 10" | 43 | 6 | " | " | 16'-0" | 96 |
| 4 | " | " | 8" | 17 | 3 | Lat. Bracing | " | 8'-0" | 24 |
| 4 | Bent Plates | 8x⅝" | 2'-2" | 89 | 6 | " " | " | 12'-0" | 72 |
| 4 | Channels | 12"x20½# | 1'-1" | 44 | 4 | " " | " | 14'-0" | 56 |
| 8 | Dowels | ¾"φ | 8" | 4 | 7 | Bridging | 2x4 | 16'-0" | 75 |
| 44 | Cut Washers | 2½"x¼" | – | 28 | | | | | |
| | Nails | 16d | | 7 | | | | | |
| | " | 20d | | 19 | | | | | |
| | " | 50d | | 107 | | | | | |

FIG. 6.—TIMBER TRUSS BRIDGE, IOWA HIGHWAY COMMISSION.

1,500–$25l/d$ where $l$ = length, and $d$ = least side or diameter, both in inches. Axial tension on net section of steel, 16,000.

Details of a combination timber bridge with a span of 40 ft. are given in Fig. 7. A hip casting was used as shown. The end shoe is made of a 12″ × 12″ × ½″ plate to which is riveted a 3½″ × 5″ × ½″ angle 12 in. long.

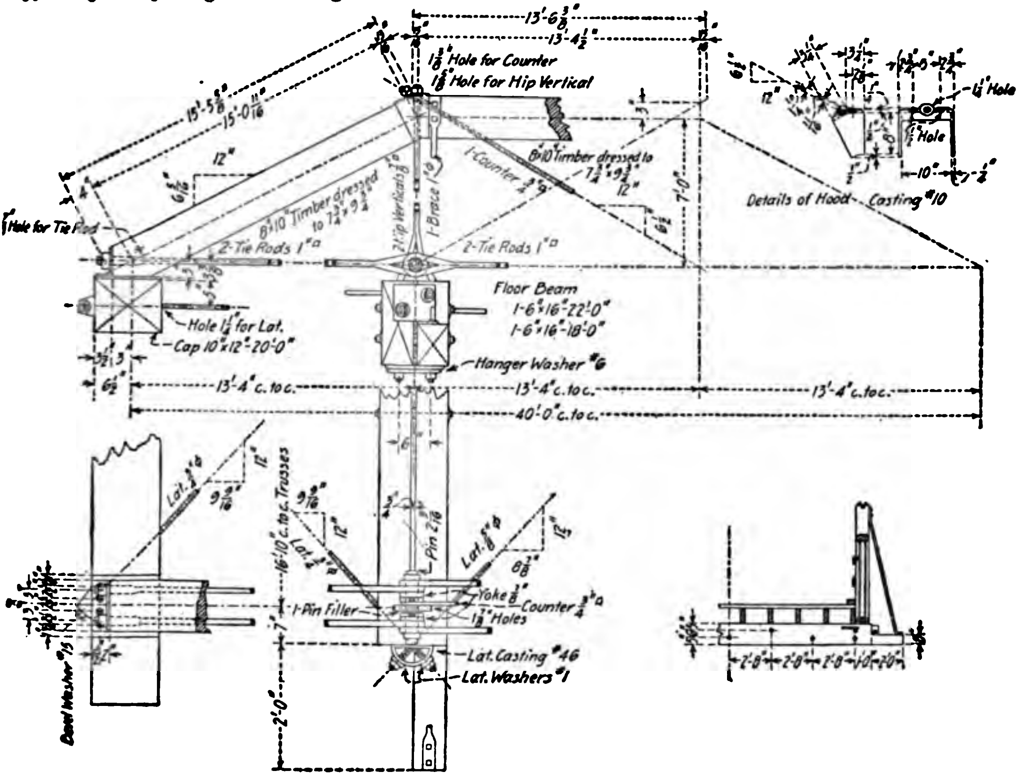



FIG. 7. DETAILS OF A TIMBER HIGHWAY BRIDGE.

Details of a 100-ft. span combination bridge are shown in Fig. 8. Castings are used for the shoe, hip and upper chord joints, while steel shoes are used for the intermediate posts. This bridge may also be built with steel floorbeams. The weight of the metal in a combination bridge will vary with the details. For the bridge shown in Fig. 8, the total weight of the cast iron and steel is about 45 per cent of the weight of a steel bridge having the same capacity.

The details of a 60-ft. span Howe truss highway bridge built in 1916 by the Utah Highway Commission are shown in Fig. 9. This bridge is designed for a dead load of 680 lb. per lineal foot, and a live load of 100 lb. per sq. ft. and an 18-ton road roller. Impact on trusses 18 per cent, on floor 25 per cent. The tension in the lower chord is carried by a steel plate laid on top of two 4 in. × 4 in. timbers. The trusses are braced laterally by means of 3 in. × 3 in. × ⅜ in. angles. This bridge contains 14,407 board feet of lumber, 4,247 lb. of steel angles, plates, bolts and nails, 1,856 lb. of castings, and 729 lb. of hangers and nuts. Bridges of this type are built with spans from 20 ft. to 80 ft.

**DETAILS OF DESIGN.**—The allowable stresses to be used in designing timber bridges and trestles are given in the General Specifications for Timber Bridges and Trestles in the latter

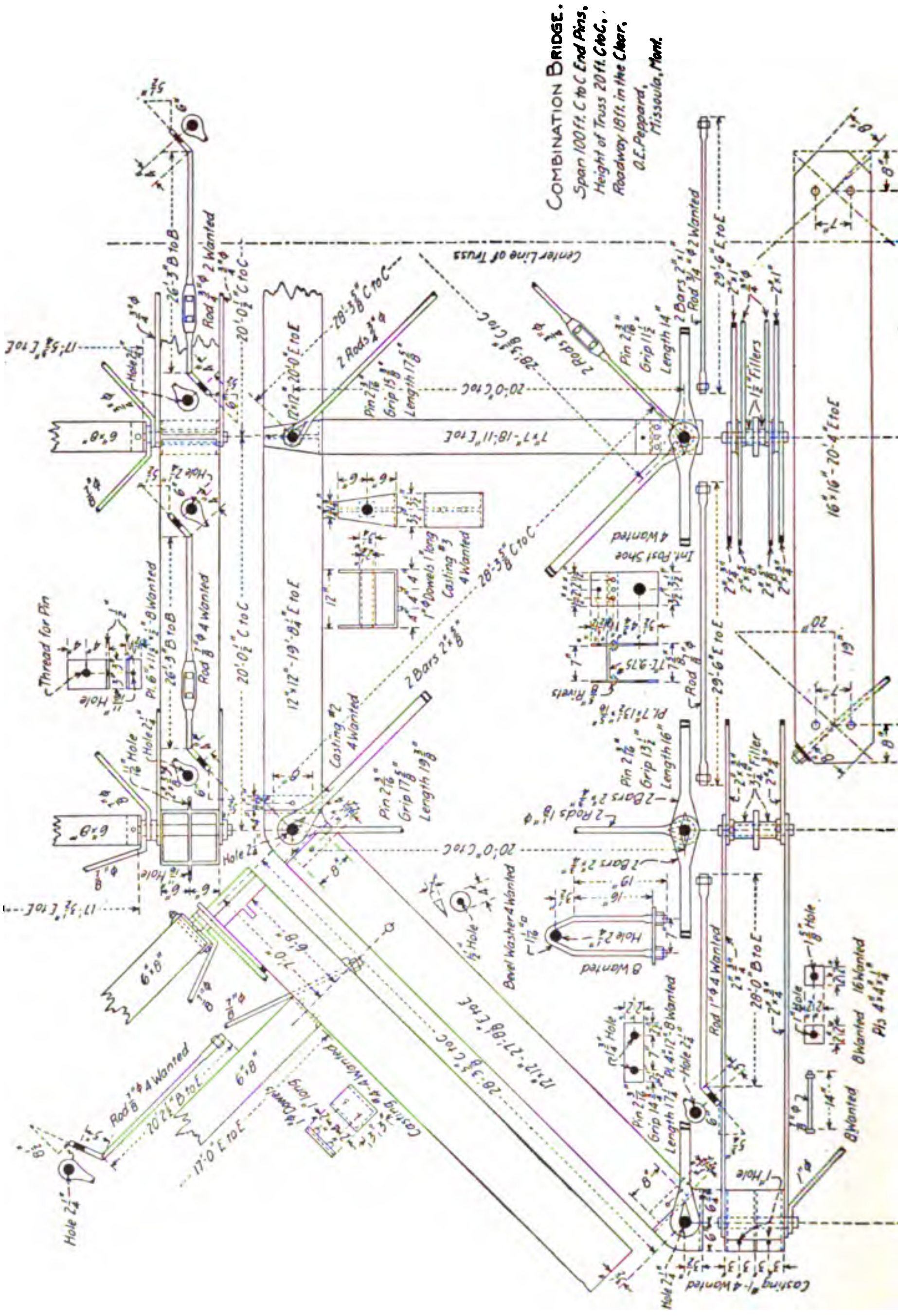



FIG. 8. COMBINATION HIGHWAY BRIDGE.

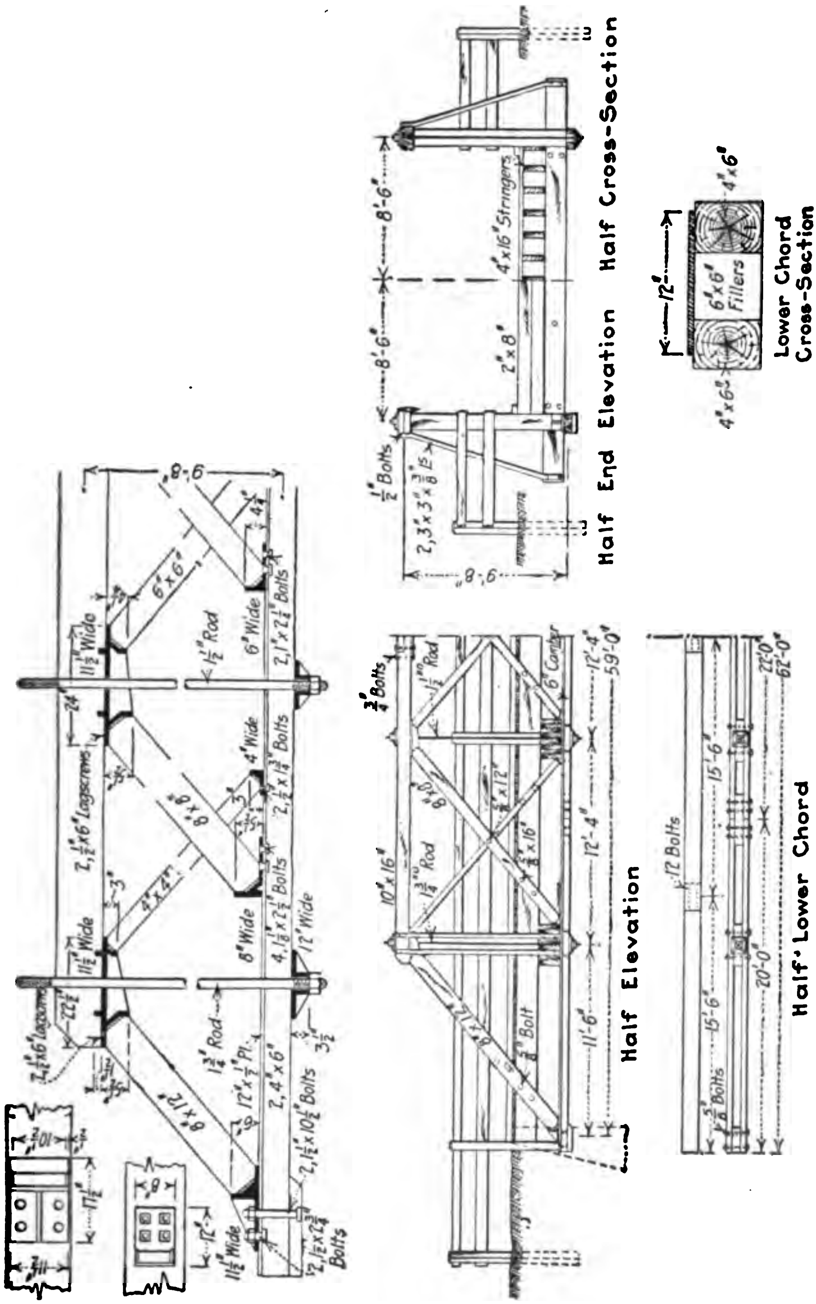



FIG. 9. TIMBER HOWE TRUSS BRIDGE. UTAH HIGHWAY COMMISSION.

part of this chapter. The allowable stresses used by the Iowa Highway Commission in the design of timber bridges and trestles are given in the description of the standard truss bridges designed by the Commission. The effect of impact is usually omitted in designing timber bridges.

**Pressure on Inclined Surfaces.**—The safe unit compressive stress on timber on surfaces inclined to the direction of the fibers may be calculated by the formula proposed by Professor H. S. Jacoby, which is as follows:

$$p = e \cdot \sin^2 \theta + c \cdot \cos^2 \theta$$

where $p$ = allowable unit compression in lb. per sq. in. on inclined surface;
$e$ = allowable bearing in lb. per sq. in. parallel to fibers;
$c$ = allowable bearing in lb. per sq. in. perpendicular to fibers;
$\theta$ = angle between surface and fibers of timber.

**Lateral Strength of Wire Nails.**—The safe lateral resistance of wire nails may be taken as

$$p = 8d$$

where $p$ is safe working resistance of one nail in pounds, and $d$ is the penny designation of the nail.

**Lag Screws.**—The safe lateral resistance of lag screws when fastening plates $\frac{1}{4}$ to $\frac{1}{2}$ in. thick to timbers may be taken as follows:

$\frac{5}{8}$ in. $\times$ 4 in. lag screws, 1,200 lb.
$\frac{3}{4}$ in. $\times$ $4\frac{1}{2}$ in. lag screws, 1,500 lb.

When fastening planking to timbers where the thickness of the planking is not greater than $\frac{3}{8}$ the length of lag screw, reduce each of above values 200 lb.

**Design of Bolts.**—Bolts when used in making timber splices should have full size washers and should be well drawn up. In splicing timber the splice bolts take bending stresses as well as shearing stresses. For timbers not more than two inches thick the bolts may be designed for shear only. Professor H. S. Jacoby in "Structural Details" assumes the bearing stress on the bolt as uniform for its entire length and shows that the bending moment in the bolt equals the stress in the splice timber multiplied by the distance from the center of the splice timber to the quarter point of the main timber. For a main timber with a width $b$ and carrying a stress $P$, spliced with two splice timbers with a width $b/2$, the total bending moment on the splice bolts will be

$$M = \frac{P}{2} \times \left(\frac{b}{4} + \frac{b}{4}\right) = \frac{P \cdot b}{4}$$

The resisting moment of a bolt is equal to

$$M' = f \cdot \pi \cdot d^3/32$$

where $f$ is the allowable bending stress in the bolt and $d$ is the diameter of the bolt in inches.

**Safe Bearing on Bolts.**—The bearing surface is partly inclined to the direction of the stress and the safe bearing unit stress on a round bolt will be less than for a square bolt. Professor Jacoby shows that if the ratio of safe bearing on the end fibers to the safe bearing on the sides of the fibers is 0.25, the safe bearing stress of a round bolt when used as a splice bolt will be 0.62 or $\frac{5}{8}$ of the safe bearing on a square bolt.

## Definitions of Timber Bridges and Trestles.

**Definitions.**—The following definitions have been adopted by the American Railway Engineering Association.

**Wooden Trestle.**—A wooden structure composed of upright members supporting simple horizontal members or beams, the whole forming a support for loads applied to the horizontal members.

**Frame Trestle.**—A structure in which the upright members or supports are framed timbers.

**Pile Trestle.**—A structure in which the upright members or supports are piles.

**Bent.**—The group of members forming a single vertical support of a trestle, designated as pile bent where the principal members are piles, and as framed bent where of framed timbers.

**Post.**—One of the vertical or battered members of the bent of a framed trestle.

**Pile.**—(See definition under subject of Piles and Pile Driving.)

**Batter.**—A deviation from the vertical in upright members of a bent.

**Cap.**—A horizontal member upon the top of piles or posts, connecting them in the form of a bent.

**Sill.**—A lower horizontal member of a framed bent.

**Sub-Sill.**—A timber bedded in the ground to support a framed bent.

**Intermediate Sill.**—A horizontal member in the plane of the bent between the cap and sill to which the posts are framed.

**Sway Brace.**—A member bolted or spiked to the bent and extending diagonally across its face.

**Longitudinal Strut or Girt.**—A stiff member running horizontally, or nearly so, from bent to bent.

**Longitudinal X-Brace.**—A member extending diagonally from bent to bent in a vertical or battered plane.

**Sash Brace.**—A horizontal member secured to the posts or piles of a bent.

**Stringer.**—A longitudinal member extending from bent to bent and supporting the ties.

**Jack Stringer.**—A stringer placed outside of the line of main stringers.

**Tie.**—A transverse timber resting on the stringers and supporting the rails.

**Guard Rail.**—A longitudinal member, usually a metal rail, secured on top of the ties inside of the track rail, to guide derailed car wheels.

**Guard Timber.**—A longitudinal timber framed over the ties outside of the track rail, to maintain the spacing of the ties.

**Packing Block.**—A small member, usually wood, used to secure the parts of a composite member in their proper relative positions.

**Packing Spool or Separator.**—A small casting used in connection with packing bolts to secure the several parts of a composite member in their proper relative positions.

**Drift Bolt.**—A piece of round or square iron of specified length, with or without head or point, driven as a spike.

**Dowel.**—An iron or wooden pin, extending into, but not through, two members of the structure to connect them.

**Shim.**—A small piece of wood or metal placed between two members of a structure to bring them to a desired relative position.

**Fish-Plate.**—A short piece lapping a joint, secured to the side of two members, to connect them end to end.

**Bulkhead.**—A wall of timber placed against the side of an end bent to retain the embankment.

## Structural Timber.

**Definitions.**—The following definitions have been adopted by the American Railway Engineering Association.

**Timber.**—A single stick of wood of regular cross-section.

**Cross-Section.**—A section of a stick at right angles to the axis.

**True.**—Of uniform cross-section. Defects are caused by wavy or jagged sawing or consist of trapezoidal instead of rectangular cross-sections.

**Axis.**—The line connecting the centers of successive cross-sections of a stick.

**Straight.**—Having a straight line for an axis.

**Out of Wind.**—Having the longitudinal surfaces plane.

**Full Length.**—Long enough to "square" up to the length specified in the order.

**Corner.**—The line of intersection of the planes of two adjacent longitudinal surfaces.

**Girth.**—The perimeter of a cross-section.

**Side.**—Either of the two wider longitudinal surfaces of a stick.

**Edge.**—Either of the two narrower longitudinal surfaces of a stick.

**Face.**—The surface of a stick which is exposed to view in the finished structure.

**Sapwood.**—A cylinder of wood next to the bark and of lighter color than the wood within. It may be of uneven thickness.

**Heartwood.**—The older and central part of a log, usually darker in color than sapwood. It appears in strong contrast to the sapwood in some species, while in others it is but slightly different in color.

**Springwood.**—The inner part of the annual ring formed in the earlier part of the season, not necessarily in the spring, and often containing vessels or pores.

**Summerwood.**—The outer part of the annual ring formed later in the season, not necessarily in the summer, being usually dense in structure and without conspicuous pores.

**Decay.**—Complete or partial disintegration of the cell walls, due to the growth of fungi.

**Sound.**—Free from decay.

**Solid.**—Without cavities; free from loose heart, wind shakes, bad checks, splits or breaks, loose slivers, and worm or insect holes.

**Wane.**—A deficient corner due to curvature or to taper of the log.

**Square Cornered.**—Free from wane.

**Knot.**—The hard mass of wood formed in a trunk at a branch, with the grain distinct and separate from the grain of the trunk.

**Cross-Grain.**—The gnarly mass of wood surrounding a knot, or grain injuriously out of parallel with the axis.

**Wind Shake.**—A crack or fissure, or a series of them, caused during growth.

## STANDARD DEFECTS OF STRUCTURAL TIMBER.*

The standard defects included in the following list are mostly such as may be termed natural defects, as distinguished from defects of manufacture. The latter have usually been omitted, because the defects of manufacture are of minor significance in the grading of structural timber:

**Sound Knot.**—A sound knot is one which is solid across its face and is as hard as the wood surrounding it. It may be either red or black, and is so fixed by growth or position that it will retain its place in the piece.

**Loose Knot.**—A loose knot is one not firmly held in place by growth or position.

**Pith Knot.**—A pith knot is a sound knot with a pith hole not more than ½ in. in diameter† in the center.

**Encased Knot.**—An encased knot is one which is surrounded wholly or in part by bark or pitch. Where the encasement is less than ½ in. in width on each side, nor exceeding one-half the circumference of the knot, it shall be considered a sound knot.

**Rotten Knot.**—A rotten knot is one not as hard as the wood surrounding it.

**Pin Knot.**—A pin knot is a sound knot not over ½ in. in diameter.

**Standard Knot.**—A standard knot is a sound knot not over 1½ in. in diameter.

**Large Knot.**—A large knot is a sound knot, more than 1½ in. in diameter.

**Round Knot.**—A round knot is one which is oval or circular in form.

**Spike Knot.**—A spike knot is one sawn in a lengthwise direction. The mean or average diameter shall be taken as the size of these knots.

**Pitch Pockets.**—Pitch pockets are openings between the grain of the wood, containing more or less pitch or bark. These shall be classified as small, standard and large pitch pockets.

*Small Pitch Pocket.*—(a).—A small pitch pocket is one not over ⅛ in. wide.

*Standard Pitch Pocket.*—(b).—A standard pitch pocket is one not over ⅜ in. wide nor over 3 in. in length.

*Large Pitch Pocket.*—(c).—A large pitch pocket is one over ⅜ in. wide, or over 3 in. in length.

**Pitch Streak.**—A pitch streak is a well-defined accumulation of pitch at one point in the piece. When not sufficient to develop a well-defined streak, or where the fiber between grains, that is, the coarse grained fiber, usually termed "spring wood," is not saturated with pitch, it shall not be considered a defect.

**Shakes.**—Shakes are splits or checks in timber which usually cause a separation of the wood between annual rings.

**Ring Shake.**—An opening between annual rings.

**Through Shakes.**—A shake which extends between two faces of a timber.

**Rot, Dote and Red Heart.**—Any form of decay which may be evident either as a dark red discoloration not found in the sound wood, or by the presence of white or red rotten spots, shall be considered as a defect.

**Wane.**—(See definition under the subject of Structural Timber.)

*Note.*—See additional definitions of defects under Structural Timber.

## PILES AND PILE DRIVING.‡

The following definitions and the principles of Pile Driving have been adopted by the American Railway Engineering Association.

**Pile.**—A member usually driven or jetted into the ground and deriving its support from the underlying strata, and by the friction of the ground on its surface. The usual functions of a pile are: (a) to carry a superimposed load; (b) to compact the surrounding ground; (c) to form a wall to exclude water and soft material, or to resist the lateral pressure of adjacent ground.

**Head of Pile.**—The upper end of a pile.

* Adopted by Am. Ry. Eng. Assoc., Vol. 8, 1907.

† Measurements which refer to the diameter of knots or holes shall be considered as the mean or average diameter in all cases.

‡ For an elaborate bibliography on "Piles and Pile Driving" see Am. Ry. Eng. Assoc., Vol. 10.

**Head of Pile.**—The upper end of a pile.

**Foot of Pile.**—The lower end of a pile.

**Tip of Pile.**—The smaller end of a pile.

**Bearing Pile.**—One used to carry a superimposed load.

**Screw Pile.**—One having a broad-bladed screw attached to its foot to provide a larger bearing area.

**Disc Pile.**—One having a disc attached to its foot to provide a larger bearing area.

**Batter Pile.**—One driven at an inclination to resist forces which are not vertical.

**Sheet Pile.**—Piles driven in close contact in order to provide a tight wall, to prevent leakage of water and soft materials, or driven to resist the lateral pressure of adjacent ground.

**Pile Driver.**—A machine for driving piles.

**Hammer.**—A weight used to deliver blows to a pile to secure its penetration.

**Drop Hammer.**—One which is raised by means of a rope and then allowed to drop.

**Steam Hammer.**—One which is automatically raised and dropped a comparatively short distance by the action of a steam cylinder and piston supported in a frame which follows the pile.

**Leads.**—The upright parallel members of a pile driver which support the sheaves used to hoist the hammer and piles, and which guide the hammer in its movement.

**Cap.**—A block used to protect the head of a pile and to hold it in the leads during driving.

**Ring.**—A metal hoop used to bind the head of a pile during driving.

**Shoe.**—A metal protection for the point or foot of a pile.

**Follower.**—A member interposed between the hammer and a pile to transmit blows to the latter when below the foot of the leads.

**PILE-DRIVING—Principles of Practice.**—(1) A thorough exploration of the soil by borings, or preliminary test piles, is the most important prerequisite to the design and construction of pile foundations.

(2) The cost of exploration is frequently less than that otherwise required merely to revise the plans of the structure involved, without considering the unnecessary cost of the structures due to lack of information.

(3) Where adequate exploration is omitted, it may result in the entire loss of the structure, or in greatly increased cost.

(4) The proper diameter and length of pile, and the method of driving depend upon the result of the previous exploration and the purpose for which they are intended.

(5) Where the soil consists wholly or chiefly of sand, the conditions are most favorable to the use of the water jet.

(6) In harder soils containing gravel the use of the jet may be advantageous, provided sufficient volume and pressure be provided.

(7) In clay it may be economical to bore several holes in the soil with the aid of the jet before driving the pile, thus securing the accurate location of the pile, and its lubrication while being driven.

(8) In general, the water jet should not be attached to the pile, but handled separately.

(9) Two jets will often succeed where one fails; in special cases a third jet extending a part of the depth aids materially in keeping loose the material around the pile.

(10) Where the material is of such a porous character that the water from the jets may be dissipated and fail to come up in the immediate vicinity of the pile, the utility of the jet is uncertain, except for a part of the penetration.

(11) A steam or drop hammer should be used in connection with the water jet, and used to test the final rate of penetration.

(12) The use of the water jet is one of the most effective means of avoiding injury to piles by overdriving.

(13) There is danger from overdriving when the hammer begins to bounce. Overdriving is also indicated by the bending, kicking or staggering of the pile.

(14) The brooming of the head of a pile dissipates a part, and in some cases all, of the energy due to the fall of the hammer.

(15) The weight or the drop of the hammer should be proportioned to the weight of the pile, as well as to the character of the soil to be penetrated.

(16) The steam hammer is more effective than the drop hammer in securing the penetration of a pile without injury, because of the shorter interval between blows.

(17) Where shock to surrounding material is apt to prove detrimental to the structure, the steam hammer should always be used instead of the drop hammer. This is especially true in the case of sheet piling which is intended to prevent the passage of water. In some cases also the jet should not be used.

(18) In general, the resistance of piles, penetrating soft material, which depend solely upon skin friction, is materially increased after a period of rest. This period may be as short as fifteen minutes, and rarely exceeds twelve hours.

(19) In tidal waters the resistance of a pile driven at low tide is increased at high tide on account of the extra compression of the soil.

(20) Where a pile penetrates muck or a soft yielding material and bears upon a hard stratum at its foot, its strength should be determined as a column or beam; omitting the resistance, if any, due to skin friction.

(21) Unless the record of previous experience at the same site is available, the approximate bearing power may be obtained by loading test piles. The results of loading test piles should be used with caution, unless their condition is fairly comparable with that of the piles in the proposed foundation.

(22) In case the piles in a foundation are expected to act as columns the results of loading test piles should not be depended upon unless they are sufficient in number to insure their action in a similar manner, and they are stayed against lateral motion.

(23) Before testing the penetration of a pile in soft material where its bearing power depends principally, or wholly, upon skin friction, the pile should be allowed to rest for 24 hours after driving.

(24) Where the resistance of piles depends mainly upon skin friction it is possible to diminish the combined strength, or bearing capacity, of a group of piles by driving additional piles within the same area.

(25) Where there is a hard stratum overlying softer material through which the piles are to pass to a firm bearing below, the upper stratum should be removed by dredging or otherwise, provided it would injure the piles to drive through the stratum. The material removed may be replaced if it is needed to provide lateral resistance.

(26) Timber piles may be advantageously pointed, in some cases, to a 4-in. or 6-in. square at the end.

(27) Piles should not be pointed when driven into soft material.

(28) Shoes should be provided for piles when the driving is very hard, especially in riprap or shale, and should be so constructed as to form an integral part of the pile.

(29) The use of a cap is advantageous in distributing the impact of the hammer more uniformly over the head of the pile, as well as to hold it in position during driving.

(30) The specification relating to the penetration of a pile should be adapted to the soil which the pile is to penetrate.

(31) It is far more important that a proper length of pile should be put in place without injury than that its penetration should be a specified distance under a given blow, or series of blows.

## GENERAL SPECIFICATIONS FOR TIMBER BRIDGES AND TRESTLES.

BY

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1920

### Part I. Design.

1. **Classes.**—Bridges under these specifications are divided into two classes, as follows:
*Class* $D_2$. For country roads with medium traffic.
*Class* $D_3$. For country roads with light traffic.

2. **Length of Span.**—Timber trestle bridges may be used with spans of 10 to 25 ft.; timber low truss bridges with spans of 30 ft. to 60 ft.; and timber Howe high truss bridges above 60 ft.

In calculating the stresses in timber beams the length of the span shall be taken as center to center of end bearings.

3. **Width of Roadway.**—Timber bridges shall have a minimum width of roadway of 18 ft. on $D_2$ bridges, and of 16 ft. on $D_3$ bridges.

4. **Low Truss Bridges.**—Low truss bridges shall have the top chords braced by knee-braces or inclined posts at each panel point. The floorbeams shall be extended to carry the brace or inclined post.

5. **Wooden Joists.**—Wooden floor joists shall be spaced not more than 2½ ft. centers, and shall lap by each other so as to have a full bearing on the floorbeams, and shall be separated ½ in. for free circulation of air. Their width shall not be less than 3 in., or one-fourth the depth in width. The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists in feet divided by four feet. No impact shall be considered in the design of wooden joists, planks or ties. Outside joists shall be designed for the same live loads as the intermediate joists. Joists shall be designed for the allowable stresses given in Table I.

6. **Floor Plank.**—For single thicknesses the roadway planks shall not be less than 3 in. thick, nor less than one-eighth of the distance between centers of joists, and shall be laid transversely with ½ in. openings and securely spiked to each joist. All plank shall be laid with heart side down. When an additional wearing surface is required it shall be 1½ in. thick, and the lower planks of a minimum thickness of 3 in. shall be laid diagonally with ½ in. openings. All floor plank shall be spiked with three 50*d* spikes at each intersection.

7. Footwalk plank shall be not less than 2 in. thick nor more than 6 in. wide, spaced with ½ in. openings.

All plank shall be laid with heart side down, shall have full and even bearing on and be firmly attached to the joists.

8. **Wheel Guards.**—Wheel guards of a cross-section of not less than 6 in. by 4 in. shall be provided on each side of the roadway. They shall be spliced with half-and-half joints with 6 in. lap, and shall be bolted to the stringers or joist with ¾ in. bolts, spaced not to exceed 5 ft. apart.

9. **Railing.**—Unless otherwise provided on the plans, the bridge shall have a timber railing consisting of a top rail made of two pieces 2 in. by 6 in., and a lower piece 2 in., by 6 in. with posts 4 in. by 4 in. spaced not to exceed 8 ft. centers.

10. **Load on Piles.**—The maximum load on a pile shall not exceed the load given by the formula

$$P = \frac{2W \cdot h}{s + 1}$$

where $P$ = allowable load on pile in pounds; $W$ = weight of drop hammer in pounds; $h$ = height of free fall of hammer in feet; $s$ = average penetration of the pile for the last six blows of the hammer. For a steam hammer replace unity in the denominator by 0.1.

Piles shall have a penetration of not less than 10 ft. in hard ground, and not less than 15 ft. in soft ground.

11. **Dead Loads.**—The dead load shall consist of the weight of the timber and metal in the bridge and the weight of the floor and all other material.

12. **Live Loads.**—Bridges shall be designed to carry in addition to the dead load a moving load, either uniform or concentrated, or both as specified below, placed so as to give maximum stresses in each member. No allowance shall be made for impact on timber trestles or bridges.

**Class $D_2$. Ordinary Country Bridges.**—For the floor and its supports a load of 100 lb. per sq. ft. of total floor surface or a 15-ton motor truck with axles spaced 10 ft. and wheels 6-ft. centers, and occupying a space 10 ft. wide and 30 ft. long, with 10 tons on rear axle and 5 tons on front axle, with rear wheels 15 in. wide. Trusses are to be designed for a live load of 100 lb. per sq. ft. for spans of 30 ft. and less, 75 lb. per sq. ft. for spans of 80 ft. and over, and proportional between 30-ft. and 80-ft. span.

**Class $D_3$ Light Country Highway Bridges.**—For the floor and its supports a load of 90 lb. per sq. ft. of total floor surface or a 10-ton motor truck with axles spaced 10 ft. and wheels 6 ft. centers, and occupying a space 8 ft. wide by 25 ft. long, with two-thirds of total load on rear axle, with rear wheels 10 in. wide. Trusses are to be designed for a load of 90 lb. per sq. ft. for spans of 30 ft. and less, 60 lb. per sq. ft. for spans of 80 ft. and proportional between 30-ft. and 80-ft. span.

13. **Wind Loads.**—The lateral bracing in unloaded chords shall be designed for a lateral wind load of 150 lb. per lineal foot, the lateral bracing in loaded chords shall be designed for a lateral wind load of 300 lb. per lineal foot, both loads considered as a moving load. In low trusses or trestles the load on the unloaded chord may be omitted.

Trestles in addition to the above shall be designed for a lateral pressure of 100 lb. for each vertical lineal foot of trestle bent.

Wind loads need not be considered unless the wind load unit stresses are greater than 25 per cent of the unit stresses due to dead and live load loads; in which case the unit stresses due to dead and live loads may be increased 25 per cent.

## Part II. Unit Stresses.

14. **Unit Stresses.**—All parts of the structure shall be proportioned so that the sum of the maximum stresses shall not exceed the following in lb. per sq. in.

15. **Impact.**—No allowance shall be made for impact in the design of timber bridges and trestles.

16. **Timber.**—The stresses on timber shall not exceed the values given in Table I.

17. **Pressure on Inclined Surfaces.**—The safe unit compressive stress of timber on timber surfaces inclined to the direction of the fibers shall not exceed the following in lb. per sq. in.

$$p = e \cdot \sin^2 \theta + c \cdot \cos^2 \theta$$

where $p$ = allowable unit compression on inclined surface;
$e$ = allowable bearing parallel to fibers;
$c$ = allowable bearing perpendicular to fibers;
$\theta$ = angle between surface and fibers of timber.

### TABLE I.

ALLOWABLE STRESSES IN TIMBER HIGHWAY BRIDGES AND TRESTLES.
POUNDS PER SQUARE INCH.

| Kind of Timber. | Bending on Extreme Fiber. | Shear Parallel to Grain. | Long. Shear in Beams. | Compression. | | Columns. | |
|---|---|---|---|---|---|---|---|
| | | | | Perpendicular to Grain. | Parallel to Grain. Bearing. | Length Less than $15 \times d$. | Length Over $15 \times d$. |
| Douglas Fir...... | 1,500 | 210 | 140 | 400 | 1,500 | 1,200 | $1,500\left(1 - \frac{L}{60d}\right)$ |
| Longleaf Pine..... | 1,500 | 220 | 150 | 350 | 1,500 | 1,200 | $1,500\left(1 - \frac{L}{60d}\right)$ |
| Shortleaf Pine..... | 1,400 | 210 | 160 | 225 | 1,400 | 1,100 | $1,400\left(1 - \frac{L}{60d}\right)$ |
| White Pine....... | 1,100 | 125 | 90 | 200 | 1,200 | 900 | $1,200\left(1 - \frac{L}{60d}\right)$ |
| Spruce........... | 1,200 | 190 | 90 | 225 | 1,400 | 1,100 | $1,400\left(1 - \frac{L}{60d}\right)$ |
| Norway Pine..... | 1,000 | 160 | 125 | 200 | 1,000 | 750 | $1,000\left(1 - \frac{L}{60d}\right)$ |
| Western Hemlock . | 1,400 | 200 | 125 | 300 | 1,500 | 1,200 | $1,500\left(1 - \frac{L}{60d}\right)$ |
| Cypress.......... | 1,100 | 150 | 90 | 225 | 1,400 | 1,100 | $1,400\left(1 - \frac{L}{60d}\right)$ |
| White Oak....... | 1,500 | 260 | 140 | 600 | 1,500 | 1,200 | $1,500\left(1 - \frac{L}{60d}\right)$ |

$L$ = length of column in inches.
$d$ = diam. or least side of column in inches.
Stresses are for green timber and are to be used without increase for impact.
Modulus of elasticity, 1,500,000 for Douglas Fir, Longleaf Pine, Shortleaf Pine and Western Hemlock; and 1,200,000 for other timbers given in Table I.

18. **Stresses in Steel.**—Axial tension on net section, 16,000 lb. per sq. in. Axial compression on gross section, $16,000 - 70(l/r)$, lb. per sq. in. with a maximum of 14,000 lb. per sq. in., where $l$ = length and $r$ = radius of gyration of the member, both in inches.

Bending on extreme fiber of rolled or built shapes, 16,000 lb. per sq. in.; on pins 24,000 lb. per sq. in.

Shearing on shop rivets and pins, 12,000 lb. per sq. in. Shearing on field rivets and turned bolts, and on gross section of web of plate girders, 10,000 lb. per sq. in. Bearing on shop rivets and pins, 24,000 lb. per sq. in. Bearing on field rivets and turned bolts, 20,000 lb. per sq. in. Bearing on expansion steel rollers, $600d$ lb. per lineal inch, where $d$ = diameter of roller in inches. Bearing on granite and Portland cement concrete, 600 lb. per sq. in.

19. **Stresses in Cast Iron.**—Axial tension, 3,000 lb. per sq. in. Bearing, 12,000 lb. per sq. in. Shear, 1,500 lb. per sq. in. Bearing on cast iron rockers or rollers, $300d$ lb. per lineal inch, where $d$ = diameter of rocker or roller in inches.

## Part III. Materials.

20. **Timber.**—All timber shall preferably be Douglas fir, long-leaf yellow pine, white oak or western hemlock. Timber piles shall preferably be white, post or burr oak, Douglas fir, longleaf pine, tamarack, white or red cedar, chestnut, redwood or cypress.

21. **General Requirements.**—All timber shall be cut from sound live trees, and shall be sawed to standard size. It must be close grained and solid, free from defects such as injurious

ring shakes and cross grain, unsound or loose knots, knots in groups, large pitch pockets, decay or other defects that will impair its strength or fitness for the purpose intended.

22. **Size of Sawed Timber.**—All timber shall be sawed true and out of wind and shall, when dry, not measure scant in thickness more than the following.

Flooring and boards up to 1½ in. thick, may be scant $\frac{1}{16}$ in.

Planks and timbers, rough size, from 1¾ to 5¾ in. thick, may be scant ⅛ in.

Dimension timber, rough size, 6 in. thick and up, may be scant ¼ in. For example, a 12 in. × 12 in. timber may be 11¾ in. × 11¾ in.

23. **Size of Dressed Timber.**—When dressed timber more than 1½ in. in thickness is required, a reduction of ⅛ in. in thickness for each surface planed will be permitted in addition to the allowance in rough timber in § 21 and § 22. For example a 12 in. × 12 in. timber S.4S. may be 11½ in. × 11½ in.

24. **Dimension Timber.**—Dimension timber when used for chords, beams, stringers, caps, posts and sills shall show not less than 75 per cent heart on each of four faces, measured across the sides any where in the length of the piece. There shall be no loose knots, or knots greater than 2 in. in diameter, or one-quarter (¼) the width of the face of the stick in which they occur. Knots shall not be located in groups and no knot shall be nearer the edge of the stick than one-quarter (¼) the width of the face. When used for other purposes dimension timber shall be square edged with exception of 1 in. wane on one edge or ½ in. wane on two edges, and ring shakes shall not extend over one-eighth (⅛) the length of the piece.

25. **Floor Plank.**—Floor plank shall be square edged, shall show one face all heart and the other face and two edges shall show not less than seventy-five (75) per cent heart, measured across the face or sides measured anywhere in the length of the piece; and shall be free from loose knots, or sound knots more than 1½ in. in diameter.

26. **Piles.**—Timber piles shall preferably be of longleaf pine, white, burr or post oak, Douglas fir, cedar or cypress.

Piles shall be cut from sound, live trees, shall be straight, close grained and solid, free from defects such as injurious ring shakes, large and unsound or loose knots, decay or other defects that will materially impair the strength or durability. The diameter of round piles near the butt shall not be less than 12 in. nor more than 18 in., and at the tip of piles under 30 ft. not less than 8 in., nor less than 6 in. for piles more than 30 ft. long. Piles must be cut above the ground swell and must taper evenly from butt to tip. Short bends will not be allowed. A line drawn from the butt to the tip shall lie entirely within the body of the pile. All piles shall be cut square at their ends and shall be stripped of their bark.

27. **Steel.**—All structural steel shall comply with the requirements for structural steel as given in General Specifications for Steel Highway Bridges, Appendix I.

28. **Cast Iron.**—Cast iron shall comply with the requirements of the specifications of the American Society for Testing Materials for gray iron castings.

## Details of Construction.

29. **Workmanship.**—Workmanship shall be of the best quality in each class of work. Details, fastenings, and connections shall be of the best method of construction in general use on first class work.

30. Holes shall be bored for all bolts. The depth of the hole and the diameter of the auger are to be specified by the engineer.

31. Framing shall be accurately fitted; no blocking or shimming will be allowed in making joints. Timbers shall be cut off with the saw; no axe to be used. Joints and points of bearing, for which no fastening is shown on the plans, shall be fastened as specified by the engineer.

32. **Driving Piles.**—Piles shall be carefully selected to suit the place and ground where they are to be driven. When required by the eingneer, pile butts shall be banded with iron or steel for driving, and the tips with suitable iron or steel shoes, such shoes will be furnished by the contractor at actual cost. Batter piles shall be driven to the inclination shown by the plans, and shall require but slight bending before framing. Butts of all piles in a bent shall be sawed off to one plane and trimmed so as not to leave any horizontal projection outside of the cap. Piles injured in driving, or driven out of place, shall either be pulled out or cut off, and replaced by new piles.

33. **Caps.**—Caps shall be sized over the piles or posts to a uniform thickness and even bearing on piles or posts. The side with most sap shall be placed downward.

34. **Chords and Posts.**—Chords and posts shall be sawed to proper length for their position (vertical or inclined), and to an even bearing.

35. **Sills.**—Sills shall be sized at the bearing of posts to one plane.

36. **Sway Braces.**—Sway bracing shall be properly framed and securely fastened to piles or posts. When necessary for pile bents, filling pieces shall be used between the braces and the piles on account of the variation in size of piles, and securely fastened and faced to obtain a bearing against all piles.

37. **Longitudinal Braces.**—Longitudinal X-braces shall be properly framed and securely fastened to piles or posts.

38. **Girts.**—Girts shall be properly framed and securely fastened to caps, sub-sills, posts or piles, as the plans may require.

39. **Stringers.**—Stringers shall be sized to a uniform height at supports. The edges with most sap shall be placed downward.

40. **Guard Rails.**—Timber guard rails shall be framed as called for on the plans, laid to line and to a uniform top surface.

41. **Bulkheads.**—Bulkheads shall be of sufficient dimensions to keep the embankment clear of the caps and stringers at the end bents of the trestle. There shall be a space not less than two (2) in. between the back of end bent and the face of the bulkhead. The projecting ends of the bulkhead shall be sawed off to conform to the slope of the embankment, unless otherwise specified.

42. **Bolts and Rods.**—Bolts shall be of wrought-iron or steel, made with square heads, standard size, the length of thread to be 2½ times the diameter of bolt. The nuts shall be made square, standard size, with thread fitting closely the thread of bolt. Threads shall be cut according to U. S. standards. The screw ends of all rods and bars shall be upset so that the area of the section at the root of the thread shall exceed the area of the bar by 15 per cent.

43. **Drift Bolts.**—Drift bolts shall be of wrought-iron or steel, round or square, with or without head, pointed or without point, as may be called for on the plans.

44. **Spikes.**—Spikes shall be of wrought-iron or steel, square or round, as called for on the plans; steel wire spikes, when used for spiking planking, shall not be used in lengths more than 6 in.; if greater lengths are required, wrought or steel spikes shall be used.

45. **Packing Spools or Separators.**—Packing spools or separators shall be of cast-iron, made to size and shape called for on plans; the diameter of the hole shall be ⅛ in. larger than diameter of packing bolts.

46. **Cast Washers.**—Cast washers shall be of cast-iron. The diameter shall be not less than 3½ times the diameter of bolt for which the washer is used, and its thickness equal to the diameter of bolt. The hole shall be ⅛ in. larger in diameter than the diameter of the bolt.

47. **Wrought Washers.**—Wrought washers shall be of wrought-iron or steel. The diameter shall be not less than 3½ times the diameter of bolt for which the washer is used, and not less than ¼ in. thick. The hole shall be ⅛ in. larger than the diameter of the bolt.

48. **Cast Iron Bearing Blocks.**—Special castings shall be made true to pattern, without wind, free from flaws and excessive shrinkage. The size and shape are to be as called for on the plans.

49. **Painting.**—All wood railing and posts shall be given two coats of white lead paint. The painting shall be done when wood is dry, and as soon as possible after erection. The first coat shall be dry before the second coat is applied.

All truss members, and exposed ends of caps and stringers shall be given two coats of creosote oil immediately after erection. The ends of all truss members shall be given two coats of creosote oil before erection.

50. **Cleaning Up.**—On the completion of the work, all refuse material and rubbish that may have accumulated on top or under and near the trestle, by reason of its construction, shall be removed by the contractor.

**REFERENCE BOOKS.**—For additional details and data on timber bridges and trestles, the following books may be consulted.

Jacoby's "Structural Details, Framing in Heavy Timber."
Foster's "A Treatise on Wooden Trestle Bridges."
Dewell's "Timber Framing."
Ketchum's "Structural Engineers' Handbook."
"Highway Trestles, Bridges and Culverts," published by Southern Pine Association, New Orleans, La. This book covers timber bridges and is furnished gratis by the Association.
"Southern Pine Manual—Standard Wood Construction," published by Southern Pine Association, New Orleans, La.
"Structural Timber Handbook," published by West Coast Lumberman's Association, Seattle, Washington.

# PART III.

# DESIGN OF REINFORCED CONCRETE BRIDGES AND CULVERTS.

## CHAPTER XVII.

### TYPES OF REINFORCED CONCRETE BRIDGES.

**Introduction.**—Reinforced concrete highway bridges may be classed in several distinct groups as follows: (1) Slab bridges; (2) T-beam girder bridges; (3) through girder bridges; (4) deck girder bridges; (5) arch bridges; (6) pile trestles; (7) box culverts; (8) bridge culverts; and (9) circular culverts.

FIG. 1. REINFORCED CONCRETE SLAB BRIDGE WITH TIED BACK WINGS. SPAN 20 FT; ROADWAY 16 FT. ILLINOIS HIGHWAY COMMISSION.

1. **Slab Bridges.**—Slab bridges are well adapted for spans of from 10 ft. to about 25 ft. They are simple in design and give a maximum head room, and are well adapted for use in multiple span construction and for use in pile trestles. A reinforced concrete slab girder bridge with a span of 20 ft. is shown in Fig. 1. Details of the tied back wing abutments are shown in Fig. 9, Chapter XX.

2. **T-beam Bridges.**—The T-beam deck bridge is adapted to spans of from 15 ft. to 30 ft., where there is more headroom than is necessary for a slab bridge. The T-beam bridge requires less materials than is required for a slab bridge.

FIG. 2. THROUGH GIRDER BRIDGE. SPAN 60 FT. ILLINOIS HIGHWAY COMMISSION.

FIG. 3. THROUGH GIRDER BRIDGE. THREE 50-FT. SPANS. ILLINOIS HIGHWAY COMMISSION.

FIG. 4. BIG VERMILLION BRIDGE NEAR LA SALLE, ILLINOIS. FOUR 45-FT. SPANS OF THROUGH REINFORCED CONCRETE GIRDERS, ONE 224-FT. STEEL SPAN; ROADWAY, 18 FT. ILLINOIS HIGHWAY COMMISSION.

3. **Deck Girder Bridge.**—The deck girder bridge is adapted to spans of from 25 ft. to about 60 ft. where headroom is not limited and to a wide roadway. It is especially adapted to viaduct construction. It may be given architectural treatment.

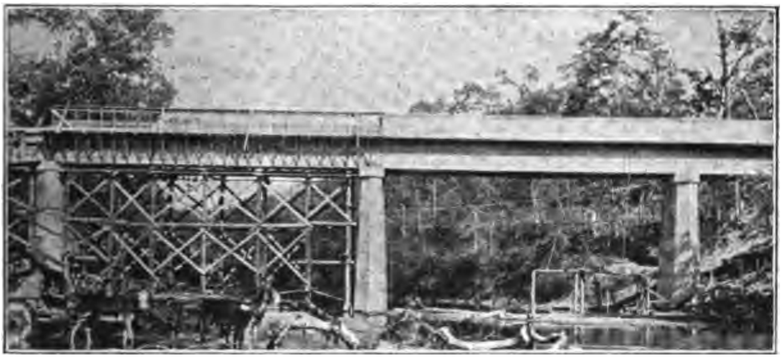


FIG. 5. DECK GIRDER BRIDGE. ILLINOIS HIGHWAY COMMISSION.

For wide bridges where headroom is adequate a combination of side through girders and interior deck girders make a very economical and satisfactory bridge. A reinforced concrete deck girder highway bridge is shown in Fig. 5.

4. **Through Girder Bridges.**—The through girder bridge is well adapted for spans of 25 ft. and up to 65 ft. where headroom is limited and the roadway is not more than 20 ft. The through girder is susceptible of architectural treatment. Through girder bridges are shown in Fig. 2, and Fig. 3.

The 60 ft. span reinforced concrete through girder bridge shown in Fig. 2, was built in Illinois in 1910 at a cost of $1,900. The reinforced concrete through girder bridge shown in Fig. 3, has three 50-ft. spans carried on reinforced concrete piers and reinforced concrete abutments. The

FIG. 6. REINFORCED CONCRETE ARCH BRIDGE. TWO 70-FT. SPANS. ROADWAY, 20 FT.

cost of this bridge built in Illinois in 1911 was $9,850. The Big Vermillion Bridge shown in Fig. 4, consists of a steel bridge of 224-ft. span, and four 45-ft. span reinforced concrete girder spans, carried on concrete piers and abutments.

5. **Arches.**—Reinforced concrete arches are adapted to clear spans more than 50 ft. to 60 ft., where solid foundations are available, the openings are high, and for structures carrying very heavy traffic loadings. Arch bridges are particularly adapted to architectural treatment. Arches are not to be recommended except where the very best foundations are available. A reinforced concrete arch bridge with two 70-ft. spans and 20-ft. roadway is shown in Fig. 6. The cost in Wisconsin in 1916 was $11,135.00.

6. **Pile Trestles.**—Reinforced concrete pile trestles are adapted to locations where a long bridge is to be carried over a low crossing, where foundation conditions are adapted to piles, and where spans of 15 ft. to 20 ft. may be used.

7. **Box Culverts.**—Reinforced concrete box culverts are well adapted to spans of from 2 ft. to 12 ft. where it is desirable to pave the channel to prevent scour.

8. **Bridge Culverts.**—Reinforced concrete bridge culverts are adapted to spans of from 6 ft. to 15 ft. where it is not desirable to pave the channel.

9. **Circular Culverts.**—Reinforced concrete circular culverts are especially adapted to openings of from 2 ft. to 4 ft. under heavy fills. The pipe may be manufactured away from the site, and in this form is adapted to a large range of conditions.

For types of reinforced concrete bridges and widths of roadway recommended by various highway commissions and other authorities, see Chapter IX.

# CHAPTER XVIII.

## Stresses in Reinforced Concrete.

**Introduction.**—The proportions, allowable stresses, materials and workmanship for reinforced concrete are given in "General Specifications for Reinforced Concrete Highway Bridges and Foundations," in Appendix II.

The stresses in reinforced concrete structures are to be calculated upon the basis of the following assumptions:

1. Calculations are to be made with reference to working stresses and safe loads.
2. A plane section before bending remains plane after bending.
3. The modulus of elasticity of concrete in compression is constant and the distribution of stresses in beams is rectilinear.
4. In calculating the moment of resistance in beams the tensile stresses in the concrete are neglected.
5. Adhesion between concrete and reinforcing steel is assumed as perfect, and concrete and steel are therefore stressed in proportion to their moduli of elasticity.
6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete is taken as $n = 15$.
7. Initial stresses in reinforcement due to the contraction of the concrete is neglected.

For the length of span to be used in calculating the stresses in reinforced concrete structures, see § 20, "General Specifications for Reinforced Concrete Highway Bridges and Foundations," Appendix II.

**STANDARD NOTATION. 1. Rectangular Beams.**

$f_s$ = tensile unit stress in steel.
$f_c$ = compressive unit stress in concrete.
$e_s$ = elongation of steel due to $f_s$.
$e_s$ = shortening of concrete due to $f_c$.
$E_s$ = modulus of elasticity of steel.
$E_c$ = modulus of elasticity of concrete.
$n = \dfrac{E_s}{E_c}$.
$M_s$ = moment of resistance relative to the steel.
$M_c$ = moment of resistance relative to the concrete.
$M$ = moment of resistance, or bending moment in general.
$A$ = steel area.
$T$ = total tension.
$C$ = total compression.
$b$ = breadth of beam.
$d$ = depth of beam to center of steel.
$k$ = ratio of depth of neutral axis to depth $d$.
$z$ = depth of resultant compression below top of beam.
$j$ = ratio of lever arm to resisting couple to depth $d$.
$j \cdot d = d - z$ = arm of resisting couple.
$p = \dfrac{A}{b \cdot d}$ = steel ratio (not percentage).
$R_s = f_s \cdot p \cdot j$ = coefficient of strength relative to steel.
$R_c = \frac{1}{2} f_c \cdot k \cdot j$ = coefficient of strength relative to concrete.

**2. T-Beams.**

$b$ = width of flange.
$b'$ = width of stem or web.
$t$ = thickness of flange.
$s = t/d$ = thickness of slab ÷ depth of beam.

**3. Beams Reinforced for Compression.**

$A'$ = area of compressive steel.
$p'$ = steel ratio for compressive steel.
$f_s'$ = compressive unit stress in steel.
$C'$ = total compressive stress in steel.
$d'$ = depth to center of compressive steel.
$z$ = depth to resultant of $C$ and $C'$.

**4. Shear and Bond.**

$V$ = total shear.
$f_v$ = maximum shearing unit stress = $V/b \cdot j \cdot d$.
$f_u$ = bond stress per unit area of bar.
$o$ = circumference or perimeter of bar.
$\Sigma o$ = sum of perimeters of bars.
$s'$ = spacing of stirrups.

**5. Columns.**

$A$ = total net area of column.
$A_s$ = area of longitudinal steel.
$A_c$ = area of concrete.
$p = \frac{A_s}{A}$ = steel ratio for longitudinal steel.
$p'$ = steel ratio of the hoops of hooped columns.
$P$ = strength of plain concrete column.
$P'$ = strength of reinforced column.
$f$ = average unit stress for entire cross-section.

**STRESSES IN RECTANGULAR BEAMS.**—In (*c*), Fig. 1, $b$ is the breadth, $d$ is the depth of the beam above the center of the reinforcing steel, $k \cdot d$ is the distance of the neutral axis below the top of the beam, and $j \cdot d$ is the arm of the resisting couple, $k$ and $j$ being ratios.

In (*a*) the deformations are shown to be proportional to the distances from the neutral axis, and in (*b*) the stress in the steel is $n$ times the stress in the concrete at the same distance from the neutral axis.

**Neutral Axis and Arm of Resisting Couple.**—Now the sum of the horizontal compressive stresses is equal to the horizontal tensile stress, and

$$\tfrac{1}{2} f_c \cdot b \cdot k \cdot d = f_s \cdot A \tag{1}$$

Substituting the value of $A = p \cdot b \cdot d$, and reducing

$$\tfrac{1}{2} f_c \cdot k = f_s \cdot p \tag{2}$$

From (*b*) Fig. 1, we have

$$f_c : f_s/n :: k \cdot d : d(1 - k),$$

and

$$f_s \cdot k \cdot d = f_c \cdot n \cdot d(1 - k)$$
$$f_s \cdot k = f_c \cdot n(1 - k) \tag{3}$$

Substituting the value of $f_s$ in (2) in (3)

$$\tfrac{1}{2}f_c \cdot k^2 = f_c \cdot p \cdot n(1 - k)$$

and

$$k^2 = 2p \cdot n(1 - k)$$
$$k = \sqrt{2p \cdot n + p^2 \cdot n^2} - p \cdot n \tag{4}$$

This formula shows that $k$ is a constant for all beams having a given percentage of reinforcement and the same grade of concrete. The values of $k$ for $n = 15$ and for different values of $p$ are given in the upper part of Fig. 2.

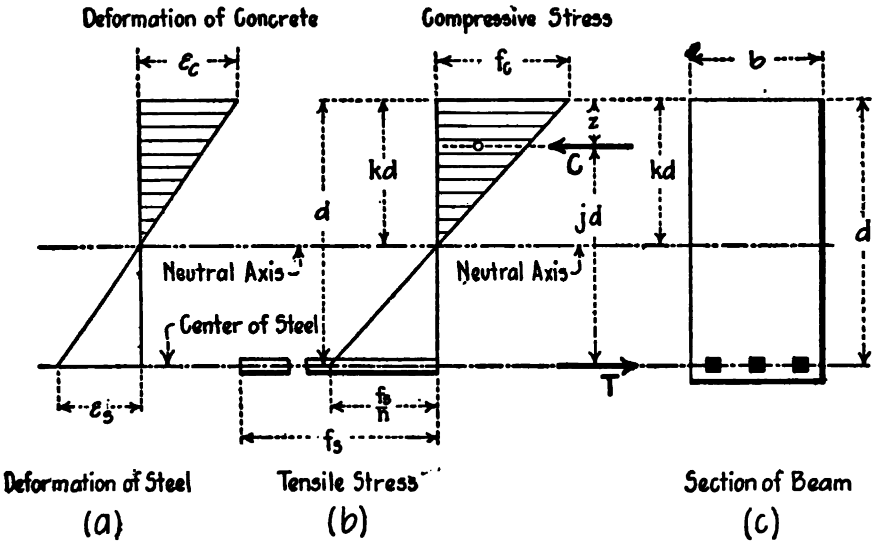



FIG. 1.

The centroid of the compressive stresses is $z = \tfrac{1}{3}k \cdot d$ below the top of the beam, and the arm of the resisting couple is

$$j \cdot d = d - \tfrac{1}{3}k \cdot d, \text{ or } j = 1 - \tfrac{1}{3}k \tag{5}$$

Values of $j$ for $n = 15$ and for different values of $p$ are given in Fig. 2. It will be seen that for $f_s = 15{,}000$ to $16{,}000$ lb. per sq. in. and $f_c = 600$ to $650$ lb. per sq. in., $j$ may be taken as $\tfrac{7}{8}$.

**Moment of Resistance.**—If the beam is under-reinforced its strength will depend on the steel, and

$$M_s = T \cdot j \cdot d = f_s \cdot A \cdot j \cdot d = f_s \cdot p \cdot j \cdot b \cdot d^2 \tag{6}$$

If the beam is over-reinforced its strength will depend on the concrete, and

$$M_c = C \cdot j \cdot d = \tfrac{1}{2}f_c \cdot b \cdot k \cdot d \cdot j \cdot d = \tfrac{1}{2}f_c \cdot k \cdot j \cdot b \cdot d^2 \tag{7}$$

The resisting moment of the beam is the smaller of the two values of $M$.

Now $R_s = f_s \cdot p \cdot j$, and $R_c = \tfrac{1}{2}f_c \cdot k \cdot j$, and equations (6) and (7) become

$$M_s = R_s \cdot b \cdot d^2 \tag{6a}$$

$$M_c = R_c \cdot b \cdot d^2 \tag{7a}$$

$$d = \sqrt{\frac{M}{R \cdot b}} \tag{6b}$$

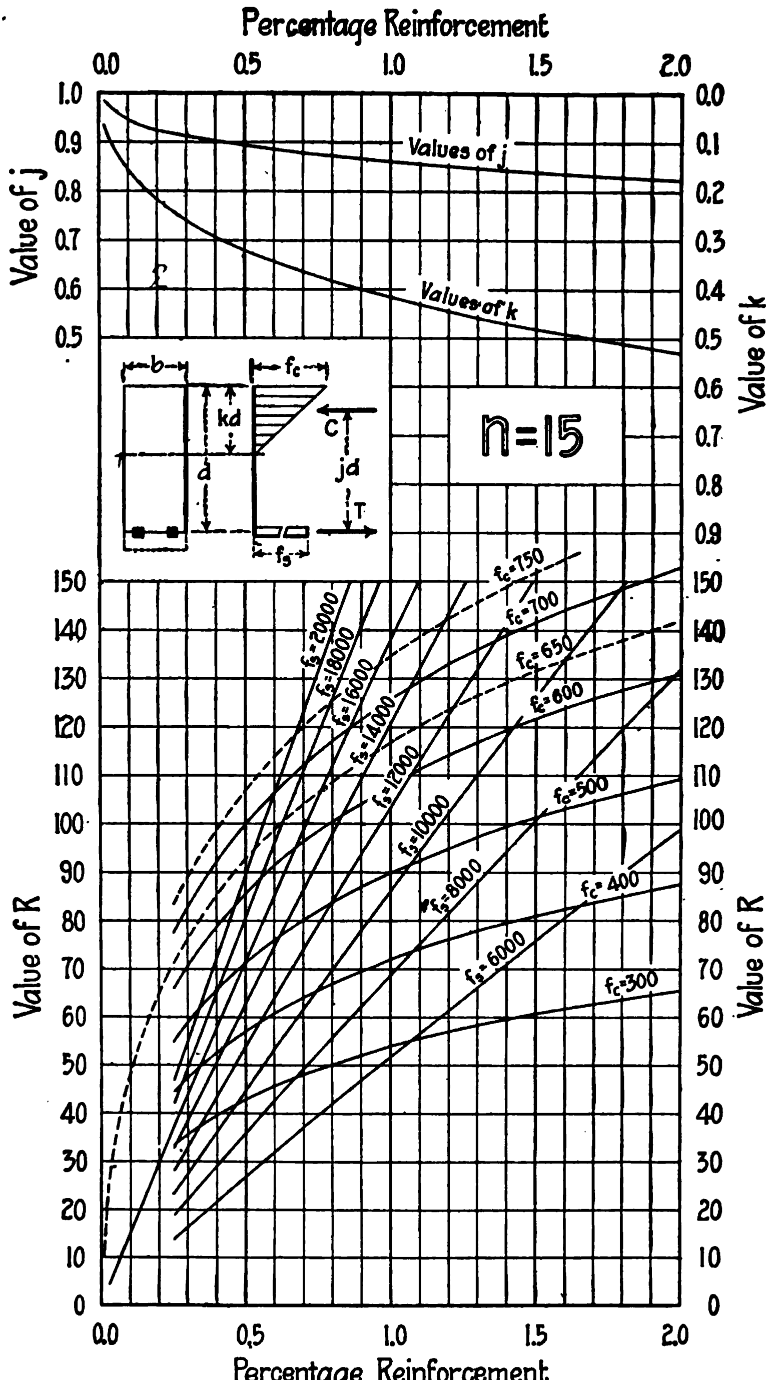

Percentage Reinforcement
Values of j
Values of k
Value of j
Value of k
n=15
b
fc
kd
C
d
jd
T
fs
fc=750
fc=700
fc=650
fc=600
fc=500
fc=400
fc=300
fs=20000
fs=18000
fs=16000
fs=14000
fs=12000
fs=10000
fs=8000
fs=6000
Value of R
Percentage Reinforcement

FIG. 2.

For $f_c = 650$ lb. and $f_s = 16{,}000$ lb. per sq. in., from Fig. 2, $R = 107.5$, and

$$d = 0.0965\sqrt{\frac{M}{b}} \tag{6c}$$

**Fiber Stresses.**—To calculate the unit fiber stresses for a given bending moment solve equations (6) and (7), and

$$f_s = \frac{T}{A} = \frac{M}{A\cdot j\cdot d} = \frac{M}{p\cdot j\cdot b\cdot d^2} \tag{8}$$

$$f_c = \frac{2M}{b\cdot k\cdot j\cdot d^2} = \frac{2f_s\cdot p}{k} \tag{9}$$

**Steel Ratio.**—If $k$ be eliminated by solving equations (2) and (3) the steel ratio will be

$$p = \frac{\frac{1}{2}}{\frac{f_s}{f_c}\left(\frac{f_s}{n\cdot f_c} + 1\right)} \tag{10}$$

If a value of $p$ less than that given by (10) is used the steel determines the strength of the beam, while if $p$ is greater the concrete will determine the strength of the beam.

**Diagram for Rectangular Beams.**—In Fig. 2 are given values of $k$ and $j$ for $n = 15$ and for different values of $p$. Values of $R_c = \frac{M}{b\cdot d^2}$ are given for different values of $f_c$ and $p$, and values of $R_s = \frac{M}{b\cdot d^2}$ are given for different values of $f_s$ and $p$. The use of the table will be shown by three problems.

**Problem 1. Moment of Resistance.**—Given the following data: $b = 10''$, $d = 20''$, $f_s = 16{,}000$ lb., $f_c = 600$ lb., 2 steel bars 1″ □ ($p = 0.01$), find $M_s$ and $M_c$.

*Solution.*—In Fig. 2 find value of percentage of reinforcement $p = 1$ per cent, on lower margin and follow the vertical line to curved line $f_c = 600$, then follow to the left on a horizontal line and find $R_c = 107$ on left margin. In like manner $R_s = 138$, which will overstress the concrete. The resisting moment will then be $M = R_c\cdot b\cdot d^2 = 107 \times 10 \times 20^2 = 428{,}000$ in.-lb.

**Problem 2. Fiber Stresses.**—Given the following data: $b = 10''$, $d = 20''$, $p = 0.009$ (0.9 per cent), $M = 360{,}000$ in.-lb., find $f_s$ and $f_c$.

*Solution.*—$R = M/b\cdot d^2 = 90$. In Fig. 2 the intersection of a vertical line through reinforcement = 0.9 per cent and a horizontal line through $R = 90$, gives $f_c = 520$ lb. and $f_s = 11{,}700$ lb.

**Problem 3. Cross-section of Beam and Percentage of Reinforcement.**—Given $M = 360{,}000$ in.-lb., $f_s = 14{,}000$ lb., $f_s = 500$ lb., to find $b$, $d$ and $p$.

*Solution.*—In Fig. 2 the intersection of curved line $f_c = 600$ lb. and straight line $f_s = 14{,}000$ lb. gives on the lower margin, $p = 0.0084$ (0.84 per cent), and on left margin gives $R = 102$. Then $b\cdot d^2 = M/R = 3{,}530$. Now if $b = 10''$, then $d = 19''$.

**STRESSES IN T-BEAMS.**—There will be two cases: (1) when the neutral axis is in the flange, and (2) when the neutral axis is in the web.

**Case I. The Neutral Axis in the Flange.**—The formulas for a rectangular beam apply, with $b$ as the flange width and $p = A \div b\cdot d$, not $A \div b'\cdot d$.

*Approximate Formulas.*—It will be seen in Fig. 3 that $j\cdot d$ is always greater than $d - t/3$, and the following formulas are on the safe side. $M_s = f_s\cdot A(d - t/3)$, and $f_s = M_s/A(d - t/3)$. There is no corresponding approximate formula for the concrete.

**Case II. The Neutral Axis in the Web.**—Where the thickness of the flange $t$ is large as compared with the depth of the beam, or as compared with the width of the web, the compression in the web may be neglected.

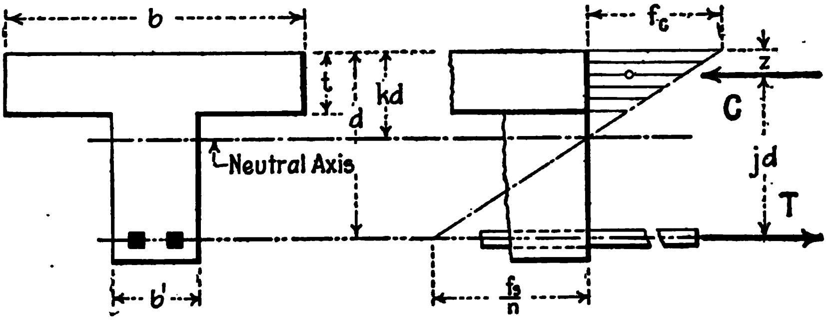



FIG. 3.

(1) **The Compression in the Web Neglected. Neutral Axis and Arm of Resisting Couple.**—As in the rectangular beam

$$f_s \cdot k = f_c \cdot n(1 - k) \tag{3}$$

and

$$k = \frac{1}{1 + f_s/n \cdot f_c} \tag{11}$$

The average unit compressive stress in the flange is

$$\tfrac{1}{2}[f_c + f_c(1 - t/k \cdot d)] = f_c\left(1 - \tfrac{1}{2}\frac{s}{k}\right),$$

and the total compression is

$$C = f_c\left(1 - \tfrac{1}{2}\frac{s}{k}\right) s \cdot b \cdot d$$

Now since $C = T$

$$f_s \cdot p \cdot b \cdot d = f_c\left(1 - \tfrac{1}{2}\frac{s}{k}\right) s \cdot b \cdot d \tag{12}$$

$$\frac{f_s}{f_c} = \left(1 - \tfrac{1}{2}\frac{s}{k}\right)\frac{s}{p} \tag{13}$$

Solving (3) and (12) for $k$ we have

$$k = \frac{p \cdot n + \frac{1}{2}s^2}{p \cdot n + s} \tag{14}$$

The arm of the resisting couple is $d - z$, where $z$ is the distance from the top of the beam to the center of the shaded area in Fig. 3.

$$z = \frac{3k - 2s}{2k - s} \times \frac{t}{3} \tag{15}$$

Also

$$j \cdot d = d - z \tag{16}$$

Substituting $k$ from (14) in (15) and $z$ from (15) in (16) we have

$$j = \frac{6 - 6s + 2s^2 + \frac{1}{2}(s^3/p \cdot n)}{6 - 3s} \tag{17}$$

**Steel Ratio in Terms of Fiber Stresses.**—Solving (13) for $p$

$$p = \left[1 - \tfrac{1}{2}\frac{s}{k}\right] s \frac{f_c}{f_s} \tag{18}$$

**Resisting Moments in Terms of Fiber Stresses.**—The resisting moment as determined by stress in steel is

$$M_s = T \cdot j \cdot d = f_s \cdot A_s \cdot j \cdot d = f_s \cdot p \cdot j \cdot b \cdot d^2 \tag{19}$$

The resisting moment as determined by stress in concrete is

$$M_c = C \cdot j \cdot d = f_c\left[1 - \tfrac{1}{2}\frac{s}{k}\right] s \cdot j \cdot b \cdot d^2 \tag{20}$$

**Coefficients of Resistance.**—By definition

$$R = \frac{M}{b \cdot d^2}$$

From (19)

$$R_s = \frac{M_s}{b \cdot d^2} = f_s \cdot p \cdot j \tag{21}$$

From (20)

$$R_c = \frac{M_c}{b \cdot d^2} = f_c\left[1 - \tfrac{1}{2}\frac{s}{k}\right] s \cdot j \tag{22}$$

**Fiber Stresses in Terms of Moment.**—Solving (19) for $f_s$

$$f_s = \frac{M}{A_s \cdot j \cdot d} = \frac{M}{p \cdot j \cdot b \cdot d^2} \tag{22a}$$

Solving (20) for $f_c$

$$f_c = \frac{M}{\left[1 - \tfrac{1}{2}\frac{s}{k}\right] s \cdot j \cdot b \cdot d^2} \tag{23}$$

**Steel Area in Terms of Steel Ratio and of Moment.**—Solving (19) for $A_s$

$$A_s = \frac{M}{f_s \cdot j \cdot d}$$

**Arm of Resisting Couple in Terms of Coefficient of Resistance.**—From (21), $p = R/f_s.j$. Substituting in (17) and solving

$$j = \frac{3(1 - s) + s^2}{3(1 - \tfrac{1}{2}s) - (f_s \cdot s^2/4n \cdot R)} \tag{24}$$

2. **Compression in Web Considered.**—Where the flange is thin as compared with the depth of the beam, $d$, and width of web, $b'$, it may become necessary to consider the compression in the web. In the same manner as in (1), we have

$$k \cdot d = \sqrt{\frac{2n \cdot d \cdot A + (b - b')t^2}{b'} + \left(\frac{n \cdot A + (b - b')t}{b'}\right)^2} - \frac{n \cdot A + (b - b')t}{b'}$$

$$z = \frac{(k \cdot d \cdot t^2 - \tfrac{2}{3}t^3)b + \left[(k \cdot d - t)^2\left(t + \dfrac{k \cdot d - t}{3}\right)\right] b'}{b \cdot t(2k \cdot d - t) + b'(k \cdot d - t)^2} \tag{25}$$

$$j \cdot d = d - z$$

$$M_s = f_s \cdot A \cdot j \cdot d$$

$$M_c = \frac{f_c}{2k \cdot d}[(2k \cdot d - t)b \cdot t + (k \cdot d - t)^2 b']j \cdot d \tag{26}$$

**Design of T-Beams.**—Where the dimensions and reinforcement of the beam are given the safe load can be calculated by the preceding formulas. If the value of $k \cdot d$ is less than $t$ the problem comes under Case I, and the formulas for rectangular beams may be used.

**STRESSES IN BEAMS REINFORCED FOR COMPRESSION.**—The beam is reinforced with steel on both the compression and tension sides.

**Neutral Axis and Arm of Resisting Couple.**—From Fig. 4, as in Fig. 1, we have

$$f_s \cdot k = f_c \cdot n(1 - k) \tag{3}$$

Also

$$f_s' \cdot k \cdot d = f_c \cdot n(k \cdot d - d'),$$

and

$$f_s' = f_c \cdot n\left(1 - \frac{d'}{k \cdot d}\right) \tag{27}$$

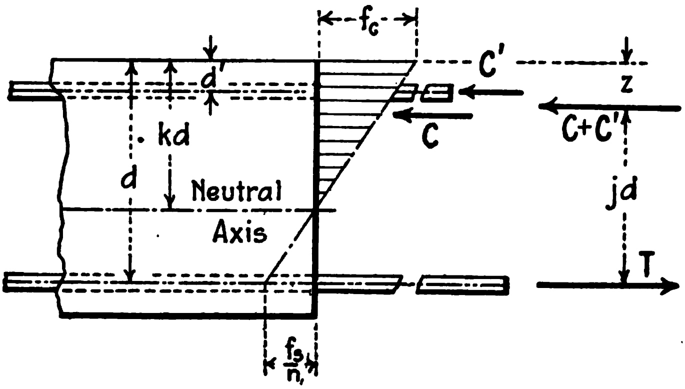



FIG. 4.

For simple flexure $T = C + C'$, and

$$f_s \cdot A = \tfrac{1}{2} f_c \cdot b \cdot k \cdot d + f_s' \cdot A' \tag{28}$$

Substituting values of $f_c$ and $f_s'$ from (3) and (27) in (28), we have

$$k^2 + 2n(p + p')k = 2n(p + p' \cdot d'/d)$$

and solving for $k$,

$$k = \sqrt{2n(p + p' \cdot d'/d) + [n(p + p')]^2} - n(p + p') \tag{29}$$

The arm of the resisting couple is

$$j \cdot d = d - z \tag{30}$$

where $z$ is given by the equation

$$z = \frac{\tfrac{1}{3}k^3 \cdot d + 2p' \cdot n \cdot d'(k - d'/d)}{k^2 + 2p' \cdot n(k - d'/d)} \tag{31}$$

**Moment of Resistance.**—If the beam is under-reinforced on the tension side the strength of the beam is determined by the steel, and

$$M_s = f_s \cdot A \cdot j \cdot d = f_s \cdot p \cdot j \cdot b \cdot d^2 \tag{32}$$

If the beam is over-reinforced on the tension side, the strength of the beam is determined by the compressive resistance and

$$M_c = \tfrac{1}{2} f_c \cdot k(1 - \tfrac{1}{3}k) b \cdot d^2 + f_s' \cdot p' \cdot b \cdot d(d - d') \tag{33}$$

If the value of $f_s'$ from (27) be substituted in (33), then

$$M_c = f_c \cdot b \cdot d^2[k(\tfrac{1}{2} - \tfrac{1}{6}k) + n \cdot p'(k - d'/d)(1 - d'/d)/k] \tag{34}$$

**Fiber Stresses.**—The stress $f_s$ for a moment $M$ is

$$f_s = \frac{M}{A \cdot j \cdot d} = \frac{M}{p \cdot j \cdot b \cdot d^2} \tag{35}$$

while the compressive stresses may be calculated by equations (3) and (27).

*Approximate Formulas.*—For approximate calculations assume that $k = 0.45$ and $j = 0.85$, and then

$$M_s = 0.85p \cdot f_s \cdot b \cdot d^2 \tag{36}$$

$$M_c = (0.19 + 10.5p')f_c \cdot b \cdot d^2 \tag{37}$$

$$f_s = 1.18M/p \cdot b \cdot d^2 \tag{38}$$

**FLEXURE AND DIRECT STRESS.**—When a member carries direct stress and at the same time acts as a beam, there are both direct stresses and bending stresses at any section. A common example is where the resultant of the external forces on a beam acting on one side of the section is not normal to the beam. There are two cases: (1) where the neutral axis is entirely outside of the beam and the combined stresses are all tension or all compression, and (2) where the neutral axis is inside the section and the stresses on the section are both tension and compression.

The following additional notation is required:

$P$ = resultant of all external forces acting on a beam on either side of the section.
$N$ = component of $P$ normal to section.
$e$ = eccentric distance of $P$.
$M$ = bending moment on section = $N \cdot e$.
$A'$ = area of steel near face most highly stressed.
$d'$ = distance from upper face to center of steel $A'$.
$A$ = area of steel near other face:
$d$ = distance from upper face to center of steel $A$.
$h$ = height of section.
$p'$ = steel ratio $A'/b \cdot h$.
$p$ = steel ratio $A/b \cdot h$.
$y$ = distance from upper face to center of the transformed section.
$A_t$ = area of the transformed section.
$I_t$ = moment of inertia of transformed section with reference to its centroidal axis.
$I_c$ = moment of inertia of the concrete with reference to the same axis.
$I_s$ = moment of inertia of the steel with reference to the same axis.

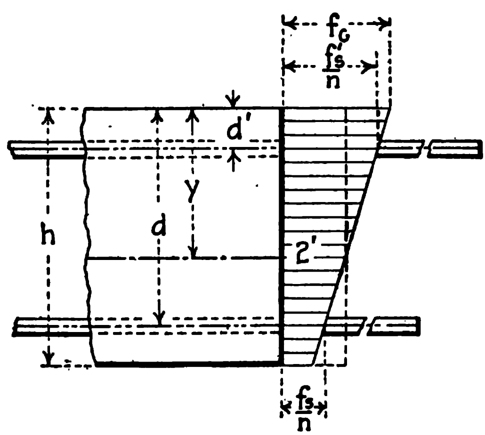



FIG. 5.

**Case I. Stresses all Compression.**—(*a*) The unit stresses in the concrete and steel can be calculated by transforming the section, the steel being assumed to be equal to concrete, having $n$ times the area of the steel, and acting with its center of gravity in the same line.

$$A_t = b\cdot h + n(A + A') \tag{39}$$

$$y = \frac{h/2 + n\cdot p\cdot d + n\cdot p'\cdot d'}{1 + n\cdot p + n\cdot p'} \tag{40}$$

$$I_c = \tfrac{1}{3}[y^3 + (h - y)^3]b \tag{41}$$

$$I_s = A(d - y)^2 + A'(y - d')^2 \tag{42}$$

$$I_t = I_c + n\cdot I_s \tag{43}$$

If the reinforcement is symmetrical and equal, $y = h/2$, and $I_c = \frac{1}{12}b\cdot h^3$, and $I_s = 2A(\frac{1}{2}h - d')^2$.

Now in Fig. 5 the direct unit stress in the concrete will be $N/A$, and the maximum flexural unit stress in the concrete is $\frac{M\cdot y}{I_t}$, and the combined stresses are

$$f_c = \frac{N}{A_t} + \frac{M\cdot y}{I_t} \tag{44}$$

$$f_s' = n\frac{N}{A_t} + \frac{n\cdot M(y - d')}{I_t} \tag{45}$$

$$f_s = n\frac{N}{A_t} - \frac{n\cdot M(d - y)}{I_t} \tag{46}$$

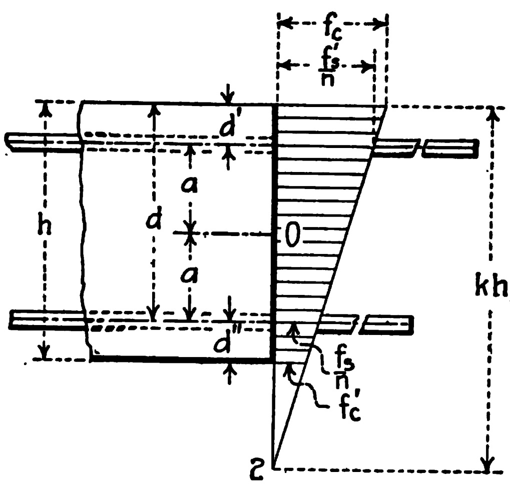



FIG. 6.

(*b*) The stresses may be calculated directly from Fig. 6 without using the transformed sections. From Fig. 6

$$f_s' = n\cdot f_c(1 - d'/k\cdot h) \tag{47}$$

$$f_s = n\cdot f_c(1 - d/k\cdot h) \tag{48}$$

$$f_c' = f_c(1 - 1/k) \tag{49}$$

Now since the resultant normal stress equals $N$, we have

$$N = \tfrac{1}{2}(f_c + f_c')b\cdot h + f_s'\cdot A' + f_s\cdot A \tag{50}$$

and since $M$ = moment of all forces about the neutral axis

$$M = \tfrac{1}{2}(f_c + f_c')b \cdot h \frac{h}{6(2k - 1)} + f_s' \cdot A' \left(\frac{h}{2} - d'\right) - f_s \cdot A \left(\frac{h}{2} - d\right) \tag{51}$$

The unit stresses may be calculated by means of the formulas above.

If the reinforcement is symmetrical, and $A = A'$, $k$ is given by the equation

$$12k(1 + 2n \cdot p)e/h = 1 + 24n \cdot p \cdot a^2/h^2 + 6(1 + 2n \cdot p)e/h \tag{52}$$

and

$$M = f_c \cdot b \cdot h^2(1 + 24n \cdot p \cdot a^2/h^2)/12k \tag{53}$$

If $e/h = \frac{1}{10}$ and $p = 1.0$ per cent, $k = 2.07$.

**Case II. Stresses, Both Tension and Compression.**—(*a*) If the tension as calculated by the formula $f_c' = \frac{N}{A} - \frac{M \cdot y}{I_t}$ does not exceed, say 60 lb. per sq. in. it will be sufficient to use the formulas of Case I.

(*b*) If the tensile stresses in the concrete are too large to be neglected the stresses may be calculated as follows:

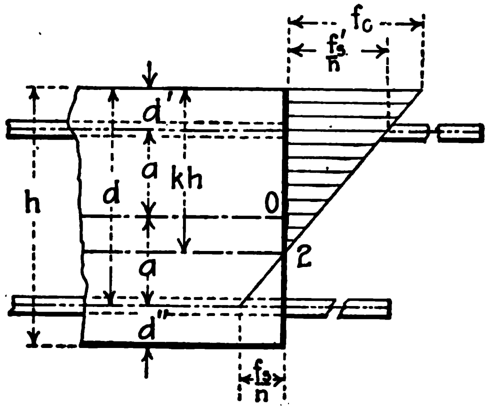



FIG. 7.

From Fig. 7 we have

$$f_s = n \cdot f_c \left(\frac{d}{k \cdot h} - 1\right) \tag{54}$$

and

$$f_s' = n \cdot f_c \left(1 - \frac{d'}{k \cdot h}\right) \tag{55}$$

The resultant fiber stress may be obtained from

$$N = \tfrac{1}{2}f_c \cdot b \cdot k \cdot h + f_s' \cdot A' - f_s \cdot A \tag{56}$$

The moment of the fiber stresses about the horizontal axis through $O$ is $M$, and

$$M = \tfrac{1}{2}f_c \cdot b \cdot k \cdot h \left(\frac{h}{2} - \frac{k \cdot h}{3}\right) + f_s' \cdot A' \left(\frac{h}{2} - d'\right) + f_s \cdot A \left(d - \frac{h}{2}\right) \tag{57}$$

If the reinforcement is equal on both sides and symmetrical we have

$$k^3 - 3\left(\frac{1}{2} - \frac{e}{h}\right)k^2 + 12n \cdot p\frac{e}{h}k = 6n \cdot p\left(\frac{e}{h} + 2\frac{a^2}{h^2}\right) \tag{58}$$

The greatest compression in the fiber is then obtained from

$$M = f_c \cdot b \cdot h^2\left[\tfrac{1}{12}k(3 - 2k) + \frac{2p \cdot n}{k}\frac{a^2}{h^2}\right] \tag{59}$$

5. **COLUMNS.**—For short columns the ratio of length to least width not exceeding 15,

$$f_s = n \cdot f_c \tag{60}$$

$$P' = f_c \cdot A_c + f_s \cdot A_s \tag{61}$$

$$= f_c \cdot A_c[1 + (n - 1)p] \tag{62}$$

$$\frac{P'}{P} = 1 + (n - 1)p \tag{63}$$

French Commission's formula for hooped columns:

$$P' = f_c \cdot A(1 + 15p + 32p') \tag{64}$$

For long columns:

$$f = \frac{f_c[1 + (n - 1)p]}{1 + \frac{1}{20{,}000}\left(\frac{l}{r}\right)^2} \tag{65}$$

Where $f$ is the average unit stress on the column, and $l$ and $r$ are the length and radius of gyration of the column respectively, both measured in the same units.

**Bond or Resistance to Slipping of Reinforcing Bars.**—Where there is no web reinforcement the shear is taken by the concrete and the shear increments are transferred to the bars by the adhesion of the concrete to the bars. The solution is the same as that for finding the pitch of rivets in the flanges of a plate girder.

Now in ($b$), Fig. 1, take two right sections at a distance $dx$ apart. Equilibrium of these two sections is maintained by the resisting moment of the bond which is equal and opposite to the moment of the vertical shear, a couple with an arm $dx$.

Taking moments about center of gravity of compressive forces we have

$$V \cdot dx = \Sigma o \cdot f_u \cdot dx \cdot j \cdot d \tag{66}$$

where $o$ = surface of bar for one inch in length and $\Sigma o$ = surface of all the bars one inch in length, $f_u$ = bond developed per square inch of surface of bar, and $V$ is the vertical shear in the beam.

Solving for $f_u$, we have

$$f_u = \frac{V}{j \cdot d \cdot \Sigma o} \tag{67}$$

Equation (67) applies to the case of horizontal bars. For inclined bars, $j \cdot d$ will be a variable and $f_u$ will be the horizontal component of the bond resistance.

**Vertical and Horizontal Shearing Stresses.**—At any point in a beam the vertical unit shearing stress is equal to the horizontal unit shearing stress. The horizontal shearing stress transmits the increments of tension to the reinforcing bars by bond stresses, as explained in the preceding discussion.

The amount of this horizontal stress transmitted to the reinforcing bars is by equation (67)

$$\Sigma o \cdot f_u = \frac{V}{j \cdot d}$$

Now if the horizontal shear just above the plane of the bars is $f_v$, the total horizontal shearing stress will be $f_v \cdot b$, which equals $\Sigma o \cdot f_u$, and

$$f_v = \frac{V}{b \cdot j \cdot d} \tag{68}$$

As an approximate formula $j$ may be taken equal to $\frac{7}{8}$, and

$$f_v = \frac{8}{7}\frac{V}{b \cdot d}$$

As no tension is assumed to exist in the concrete, the horizontal shear will be constant up to the neutral axis, above which point it decreases to zero at the top of the beam. It will be seen that

lean or poor concrete lacking in shearing strength should not be placed below the neutral axis of beams with the idea that it may be satisfactory for the reason that the concrete is assumed to take no tension.

The same formulas apply to beams reinforced for compression as regards shear and bond stress on tensile steel.

*For T-Beams.*

$$f_u = \frac{V}{j \cdot d \cdot \Sigma o} \tag{69}$$

$$f_v = \frac{V}{b' \cdot j \cdot d} \tag{70}$$

**Diagonal Tension in Concrete.**—In Mechanics of Materials (Merriman's Mechanics of Materials, p. 265, 1916 edition) it is shown that shear and tensile stresses combine to cause diagonal tensile stresses.

$$t = \tfrac{1}{2}f + \sqrt{\tfrac{1}{4}f^2 + f_v^2} \tag{71}$$

where $t$ is the diagonal tensile unit stress, $f$ is the horizontal tensile unit stress, and $f_v$ is the horizontal or vertical shearing unit stress. The direction that stress $t$ makes with the horizontal is one-half the angle whose cotangent is $\frac{1}{2}f/f_v$. If there is no tension in the concrete this reduces to

$$t = f_v \tag{72}$$

and $t$ makes an angle of 45° with the horizontal.

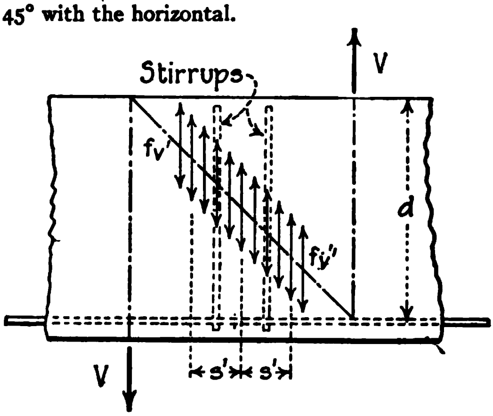



FIG. 8.

Stresses due to diagonal tension may be carried (1) by bending the reinforcing bars into a diagonal position, or (2) by means of stirrups to take the vertical component of the diagonal tension, or (3) by both bent-up bars and stirrups.

**Stresses in Stirrups.**—The following analysis is approximate but gives results that agree closely with experiments. From formula (72) it will be seen that for no tension in the concrete below the neutral axis the diagonal tension will make an angle of 45° with the horizontal; the plane of failure will then be normal to the diagonal tension and will also make an angle of 45° with the horizontal. Let $V$ be the shear in the beam not carried by the concrete. Also assume that the shear is uniform over the cross-section. Then $f_v = V/b \cdot j \cdot d = t$.

From Fig. 8, if $s'$ is the spacing of the vertical stirrups the stress in one stirrup is

$$T = f_v \cdot b \cdot s' = \frac{V}{j \cdot d} \cdot s' \tag{73}$$

Stirrups inclined at an angle of 45° will carry the diagonal tension on a section $s' \cdot b \cdot \cos 45°$. Then for diagonal stirrups

$$T = f_v \cdot b \cdot s' \cos 45° = 0.7 \frac{V}{j \cdot d} \cdot s' \qquad (74)$$

To be effective the stirrups should be spaced so that at least one stirrup will intersect the line of rupture (45° line) below the center of the beam, which requires that $s'$ never be greater than $d/2$. Rods spaced farther apart than $d$ are of no value.

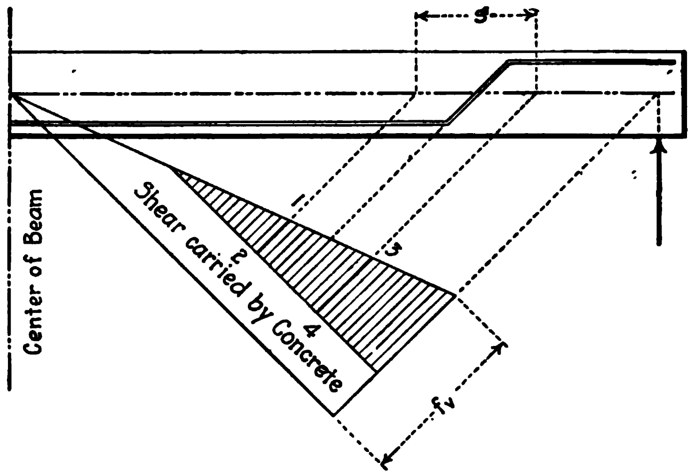



FIG. 9.

Inclined stirrups should be rigidly fastened to the horizontal reinforcement, and all stirrups should pass around the horizontal reinforcement, and have hooked ends at the top.

To calculate the stress in a bar bent up at 45° in a beam with a uniform load the method shown in Fig. 9 may be used. The shear at the support, $f_v = V/b \cdot j \cdot d$, is laid off as shown. The shear that may be carried by the concrete, is subtracted. The stress in the bent-up bar is then equal to area 1-2-3-4 $\times b$. If the bar is bent-up at some angle other than 45° the shear $f_v$ should be laid off parallel to the bent-up bar. If stirrups are used the stress carried by the bent-up bars should be subtracted before calculating stresses in the stirrups. In the specifications in Appendix II, two-thirds of the external shear may be assumed as taken by the stirrups, providing they are properly spaced. The concrete is then assumed to take one-third of the external shear, but never more than 2 per cent of the compressive strength of the concrete, or 40 lb. per sq. in. for concrete with an ultimate strength of 2,000 lb. per sq. in.

**Spacing of Bars.**—The lateral spacing of parallel bars should not be less than 3 diameters, center to center, nor should the distance from the side of the beam to the center of the nearest bar be less than 2 diameters. The clear spacing between two layers of bars should not be less than 1 inch, but the distance center to center of bars in the different layers should not be less than 3 diameters.

**T-Beams.**—In beam and slab construction, an effective bond should be provided at the junction of the beam and the slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement should be used extending over the beam and well into the slab. The slab may be considered an integral part of the beam, when adequate bond and shearing resistance between slab and web of beam is provided but its effective width should be determined by the following rules:

(*a*) It shall not exceed one-fourth of the span length of the beam.

(*b*) Its overhanging width on either side of the web shall not exceed six times the thickness of the slab.

(*c*) It must not exceed the distance between beams.

Beams in which the T-form is used only for the purpose of providing additional compression area of concrete should preferably have a width of flange not more than three times the width of the stem and a thickness of flange not less than one-third of the depth of the beam.

The ratio of $b'$ to $d$ should vary from $\frac{1}{3}$ to $\frac{1}{2}$ for small beams and may be as small as $\frac{1}{4}$ for large beams. The width $b'$ must be sufficient to provide the necessary space for tensile reinforcement.

The depth of beam in inches should be approximately equal to the span in feet.

**ABSTRACT OF REPORT OF COMMITTEE ON CONCRETE AND REINFORCED CONCRETE OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS.** The report was printed in Transactions of American Society of Civil Engineers, Vol. XLII, December, 1916.

The working stresses are given for static loads. Proper allowances are to be made for vibrations and impact. In selecting the proper working stress the designer should be guided by the working stresses used for other materials of construction, so that the entire structure may have the same degree of safety. The allowable stresses are given in terms of the ultimate compressive strength of concrete, obtained in testing concrete in cylinders 8 in. in diameter and 16 in. long, made of sluggish consistency, made and stored under laboratory conditions. The Committee recommends the following values for compressive strength of concrete to be used in design.

TABLE I.

COMPRESSIVE STRENGTH OF DIFFERENT MIXTURES OF CONCRETE, POUNDS PER SQUARE INCH.

| Aggregate. | Proportions.* | | | | |
|---|---|---|---|---|---|
| | 1:3. | 1:4½. | 1:6. | 1:7½. | 1:9. |
| Granite, trap rock | 3,300 | 2,800 | 2,200 | 1,800 | 1,400 |
| Gravel, hard limestone, and hard sandstone | 3,000 | 2,500 | 2,000 | 1,600 | 1,300 |
| Soft limestone and sandstone | 2,200 | 1,800 | 1,500 | 1,200 | 1,000 |
| Cinders | 800 | 700 | 600 | 500 | 400 |

**ALLOWABLE STRESSES.**

| | Per Cent of Compressive Strength | Lb. per Sq. In. |
|---|---|---|
| Structural steel in tension | | 16,000 |
| Concrete in compression where the surface is at least twice the loaded area | 35 | |
| Concrete for concentric compression on a plain concrete column or pier, the length of which does not exceed 4 diameters | 22.5 | |
| Compression on columns with longitudinal reinforcement only, length of the column shall not exceed 12 diameters | 22.5 | |
| (*a*) Columns with longitudinal reinforcement to the extent of not less than 1 per cent and not more than 4 per cent, and with lateral ties of not less than ¼ in. in diameter, 12 in. apart, nor more than 16 diameters of the longitudinal bar | 22.5 | |
| (*b*) Columns reinforced with not less than 1 per cent and not more than 4 per cent of longitudinal bars and with circular hoops or spirals not less than 1 per cent of the volume of the concrete and as hereinafter specified: a unit stress 55 per cent higher than given for (*a*) provided the ratio of unsupported length of column to diameter of the hooped core is not more than 10 | 34.875 | |

* Combined volume of fine and coarse aggregate measured separately.

The following limitations are placed on design of columns. Minimum size of columns 12 in. out to out. Longitudinal reinforcement to be assumed to carry its proportion of stress. Hoops or bands not assumed to carry stress. Hooping not to exceed 1 per cent of volume of column enclosed. Clear spacing of hooping not greater than one-sixth diameter enclosed column, preferably not greater than one-tenth and not more than 2½ in. Ends of hooping must be united to develop full strength.

Compression on extreme fiber of a beam, calculated for constant modulus of elasticity (stresses adjacent to the supports of continuous beams may be 15 per cent higher).... 32.5

Shear in beams with horizontal bars only and without web reinforcement................ 2

Shear in beams with vertical stirrups looped about longitudinal bars on tension side, and stirrups spaced horizontally not more than one-half the depth of the beam; or beams with longitudinal bars bent up at an angle not greater than 45° nor less than 20° with axis of beam, points of bending up spaced horizontally not more than three-quarters the depth of the beam.................................................. 4.5

Shear in beams having a combination of bent bars and vertical stirrups looped about reinforcing bars in tension side of beam and spaced horizontally not more than one-half the depth of the beam.................................................. 5

Shear in beams with web reinforcement (either vertical or inclined) securely attached to longitudinal bars in tension side of beam in such a way as to prevent slipping of bar past the stirrup, vertical stirrups spaced not more than one-half the depth of the beam, and inclined members spaced not more than three-quarters depth of beam........... 6

(The web reinforcement shall be proportioned for two-thirds the external vertical shear. The bent-up bars may be assumed as reducing the shearing stresses, but this reduction shall in no case be taken greater than 4½ per cent of compressive strength of the concrete over the effective section of the beam. When calculated by the formula $f_v = V/(b \cdot j \cdot d)$, this would mean that shear $f_v$ could not be greater than 90 lb. per sq. in. for 2,000 lb. concrete.)

The stresses in stirrups and inclined members when combined with bent-up bars are to be determined by finding the amount of the total shear that may be allowed by reason of the bent-up bars, and subtracting this shear from the total external vertical shear. Two-thirds of the remainder will be the shear to be carried by the stirrups.

The stresses in web reinforcement may be calculated by the following formulas:

Vertical web reinforcement

$$T = V' \cdot s'/j \cdot d \tag{75}$$

Bars bent up at angles between 20° and 45° with the horizontal and web members inclined at 45°

$$T = \frac{3}{4}\,\frac{V' \cdot s'}{j \cdot d} \tag{76}$$

Where $V'$ = two-thirds of the total shear producing stress in the web reinforcement; $T$ = total stress in member; $s'$ = horizontal spacing of stirrups, and $j \cdot d$ = effective depth of beam.

Punching shear, only.................................................. 6

Bond stress between concrete and plain reinforcing bars.............................. 4

Bond stress between concrete and drawn wire.............................. 2

Bond stress between concrete and deformed bars, not more than.............................. 5

The modulus of elasticity to be taken for the design as follows:

(*a*) One-fortieth that of steel, when the strength of the concrete is taken as not more than 800 lb. per sq. in.

(*b*) One-fifteenth that of steel where the strength of the concrete is taken as greater than 800 lb. per sq. in., and less than 2,200 lb. per sq. in., or less.

(*c*) One-twelfth that of steel where the strength of the concrete is taken greater than 2,200 lb. per sq. in. or less than 2,900 lb. per sq. in.

(*d*) One-tenth that of steel where the strength of concrete is taken as greater than 2,900 lb. per sq. in. In calculating deflection take one-eighth of the modulus of elasticity of steel.

**Length of Beams and Columns.**—The span length of beams and slabs simply supported should be taken as the distance center to center of supports, but need not be taken greater than the clear span plus the depth of beam or slab. For continuous or restrained beams built monolithically into the supports, the span length may be taken as the clear distance between faces of supports. Brackets should not be considered as reducing the clear span in the sense here intended, except that when brackets which make an angle of 45° or more with the axis of a restrained beam are built monolithically with the beam, the span may be measured from the section where the combined depth of the beam and the bracket is at least one-third more than the depth of the beam. Maximum negative moments are to be considered as existing at the end of the span as here defined. When the depth of a restrained beam is greater at its ends than at its mid-span and

the slope of the bottom of the beam at its ends makes an angle of not more than 15° with the direction of the axis of the beam at mid-span, the span length may be measured from face to face of supports.

The length of columns should be taken as the maximum unstayed length.

**Design of T-beams.**—In beam and slab construction an effective bond should be provided at the junction of beam and slab. When the principal reinforcement is parallel to the beam, transverse reinforcement should be used, extending over the beam and well into the slab.

The width of the slab shall not exceed one-fourth of the span length of the beam; and its overhanging width on each side of the web shall not exceed six times the thickness of the slab.

**Floor-Slabs Supported along Four Sides.**—Floor-slabs having the supports extending along the four sides should be designed and reinforced as continuous over the supports. For rectangular slabs in which the length is not greater than one and one-half times the width the portion of the total uniformly distributed load to be carried by the transverse reinforcement will be given by the formula $r = l/b - 0.5$, where $l$ = length and $b$ = width of slab. Two-thirds of the calculated moments shall be assumed as carried by the center half of the slab, and one-third by the outside quarters. The distribution of loads from slabs to the supporting beams shall be assumed as varying as the ordinates to a parabola with its vertex at the middle of the span.

**Continuous Beams and Slabs.**—When the beam or slab is continuous over its supports, reinforcement should be provided at points of negative moment. In computing bending moments for uniformly distributed loads the following rules are recommended:

(*a*) For floor-slabs, the bending moments at center and at support should be taken as $w \cdot l^2/12$ for both dead and live loads, where $w$ represents the load per linear unit and $l$ the span length.

(*b*) For beams, the bending moment at center and at support for interior spans should be taken as $w \cdot l^2/12$, and for end spans it should be taken as $w \cdot l^2/10$ for center and interior support, for both dead and live loads.

(*c*) In the case of beams and slabs continuous for two spans only, with their ends restrained, the bending moment both at the central support and near the middle of the span should be taken as $w \cdot l^2/10$.

(*d*) At the ends of continuous beams, the amount of negative moment which will be developed in the beam will depend on the condition of restraint or fixedness, and this will depend on the form of construction used. In the ordinary cases a moment of $w \cdot l^2/16$ may be taken; for small beams running into heavy columns this should be increased, but not to exceed $w \cdot l^2/12$.

For spans of unusual length, or for spans of materially unequal length, more exact calculations should be made. Special consideration is also required in the case of concentrated loads.

Even if the center of the span is designed for a greater bending moment than is called for by (*a*) or (*b*), the negative moment at the support should not be taken as less than the values there given.

**Spacing of Bars.**—Lateral spacing of parallel bars should not be less than three diameters center to center, nor two diameters from the side of the beam to the center of the bar. The clear spacing between two layers of bars should not be less than 1 in. The use of more than two layers is not recommended unless the layers are tied together by adequate metal connections, particularly at and near points where bars are bent up or down.

**Reinforcement for Temperature.**—Reinforcement not less than one-third of one per cent of a form that will develop a high bond resistance should be placed near the exposed surface and be well distributed.

## CHAPTER XIX.

### DESIGN OF RETAINING WALLS.

**Introduction.**—A retaining wall is a structure which sustains the lateral pressure of earth or some other granular mass which possesses some frictional stability. The pressure of the material supported will depend upon the material, the manner of depositing in place, and upon the amount of moisture, and will vary from zero to the full hydraulic pressure. If dry clay is loosely deposited behind the wall it will exert full pressure, due to this condition. In time the earth may become consolidated and cohesion and moisture make a solid clay, which may cause the bank to shrink away from the wall and there will be no pressure exerted. On the other hand all cohesion may be destroyed by the vibration of moving loads or by saturation, and the maximum theoretical pressures may occur. The pressures due to a dry granular mass, a semi-fluid, without cohesion, of indefinite extent, the particles held in place by friction on each other, will be considered. The effect of cohesion and of limiting the extent of the mass is considered in the author's "The Design of Walls, Bins and Grain Elevators."

**Nomenclature.**—The following nomenclature will be used:

$\phi$ = the angle of repose of the filling.
$\phi'$ = the angle of friction of the filling on the back of the wall.
$\theta$ = the angle between the back of the wall and a horizontal line passing through the heel of the wall and extending from the back into the fill.
$\delta$ = angle of surcharge, the angle between the surface of the filling and the horizontal; $\delta$ is positive when measured above and negative when measured below the horizontal.
$z$ = the angle which the resultant earth-pressure makes with a normal to the back of the wall.
$\lambda$ = the angle between the resultant thrust, $P$, and a horizontal line.
$h$ = the vertical height of the wall in feet.
$d$ = the width of the base of the wall in feet.
$b$ = the distance from the center of the base to the point where the resultant pressure, $E$, cuts the base.
$P$ = the resultant earth-pressure per foot of length of wall.
$E$ = the resultant of the earth-pressure and the weight of the wall.
$w$ = the weight of the filling per cubic foot.
$W$ = the total weight of the wall per foot of length of wall.
$p_1$ = the pressure on the foundation due to direct pressure.
$p_2$ = the pressure on the foundation due to bending moments.
$p$ = the resultant pressure on the foundation due to direct and bending forces.
$y$ = the depth of foundation below the earth surface.

**Calculation of the Pressure on Retaining Walls.**—To fully determine the pressure of the filling on a retaining wall it is necessary that the resultant of the pressure be known ($a$) in amount, ($b$) in line of action, and ($c$) in point of application. Many theories have been proposed for finding the pressure, each differing somewhat as to the assumptions and results. All theories for the design of retaining walls that have any theoretical basis come in two classes: (1) the Theory of Conjugate Pressures, due to Rankine, and commonly known as Rankine's Theory, and (2) the Theory of the Maximum Wedge, probably first proposed by Coulomb, and commonly known as Coulomb's Theory. Rankine's Theory determines the thrust in amount, in line of action, and in point of application. In Coulomb's Theory, with the exception of Weyrauch's solution, the line of action and point of application must be assumed, thus leading to numerous solutions of

more or less merit. All solutions based on the theory of the wedge assume that the resultant thrust is applied at one-third the height for a wall with a level or inclined surcharge, as is given by Rankine; but the resultant is assumed as making angles with a normal to the back of the wall varying from zero to the angle of repose of the filling. In Rankine's solution the resultant pressure is parallel to the plane of the surcharge for a vertical wall with a level or positive surcharge.

(1) **RANKINE'S THEORY.**—In this theory the filling is assumed to consist of an incompressible, homogeneous, granular mass, without cohesion, the particles are held in position by friction on each other; the mass being of indefinite extent, having a plane top surface, resting on a homogeneous foundation, and being subjected to its own weight. The principal and conjugate stresses in the mass are calculated, thus leading to the ellipse of stress. In the analysis it is proved (*a*) that the maximum angle between the pressure on any plane and the normal to the plane is equal to the angle of internal friction, and (*b*) that there is no active upward component of stress in a granular mass. Both of these laws have been verified by experiments on semi-fluids. Rankine deduced algebraic formulas for calculating the resultant pressure on a vertical wall with a horizontal surcharge, and on a vertical wall with a surcharge equal to $\delta$, an angle equal to or less than the angle of repose. The general case is best solved by constructing the ellipse of stress by graphics, or Weyrauch's algebraic solution may be used. The author has extended Rankine's solution in "The Design of Walls, Bins and Grain Elevators," so that it is perfectly general.

**Rankine's Formulas.**—With a vertical wall and a horizontal surcharge, Fig. 1, the total resultant pressure is

$$P = \tfrac{1}{2}w\cdot h^2\,\frac{1 - \sin\phi}{1 + \sin\phi} \tag{1}$$

where $w$ is the weight of the filling in lb. per cu. ft., $h$ is the depth of the wall in feet, $\phi$ is the angle of repose of the filling, and $P$ is the resultant pressure on the wall in pounds. The resultant pressure, $P$, will be horizontal.

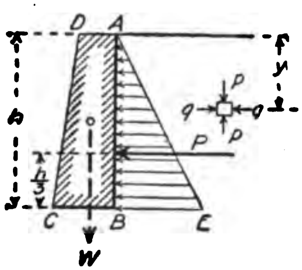



FIG. 1.

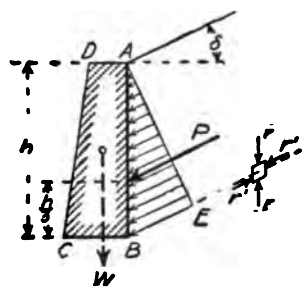



FIG. 2.

For a vertical wall with surcharge at an angle $\delta$, Fig. 2, the pressure is given by the formula

$$P = \tfrac{1}{2}w\cdot h^2\cdot\cos\delta\,\frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}} \tag{2}$$

Where $\delta$ is equal to $\phi$, formula (2) becomes

$$P = \tfrac{1}{2}w\cdot h^2\cos\phi \tag{3}$$

The resultant pressure, $P$, is parallel to the inclined top surface for a vertical wall with a level or a positive surcharge (many authors have incorrectly assumed that the resultant pressure is always parallel to the top surface of the surcharged filling).

**Inclined Retaining Wall.**—The pressure on an inclined retaining wall may be calculated by means of the ellipse of stress—see the author's "The Design of Walls, Bins and Grain Elevators."

The pressure on an inclined retaining wall may also be calculated by means of the graphic solution shown in Fig. 3 if the direction of the thrust be known. From Rankine's theory we know that the resultant pressure on a vertical retaining wall is always parallel to the top surface where the surcharge is level or is inclined upwards away from the wall. The pressure on a retaining wall inclined away from the filling may then be calculated as follows:

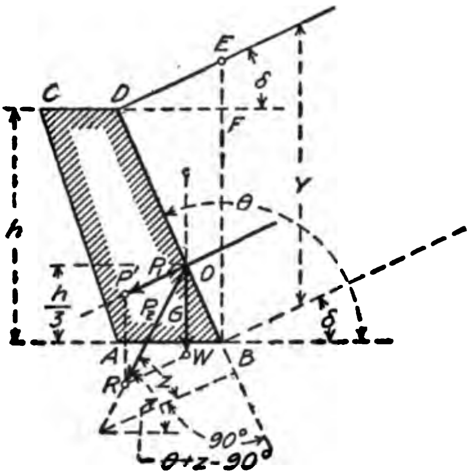



FIG. 3. PRESSURE ON AN INCLINED RETAINING WALL.

In Fig. 3 the retaining wall *ACDB* sustains the pressure of a filling having an angle of repose $\phi$, and sloping up away from the top of the wall at an angle $\delta$. Calculate $P'$ the pressure on the plane *E–B* by means of formula (2). $P'$ acts at a point $\frac{1}{3}EB$ above *B* and is parallel to the top surface *DE*. Let the weight of the triangle of filling *DBE* be *G*, which acts through the center of gravity of the triangle and intersects $P'$ at point *O*. Then $P_2$, the resultant of $P'$ and *G*, will be the resultant pressure at *O*, and makes an angle $z$ with a normal to the back of the wall, and an angle, $\lambda = \theta + z - 90°$ with the horizontal.

(2) **COULOMB'S THEORY.**—In this theory it is assumed that there is a wedge having the wall as one side and a plane called the plane of rupture as the other side, which exerts a maximum thrust on the wall. The plane of rupture lies between the angle of repose of the filling and the back of the wall. It may coincide with the plane of repose. For a wall without surcharge (horizontal surface back of the wall) and a vertical wall the plane of rupture bisects the angle between the plane of repose and the back of the wall. This theory does not determine the direction of the thrust, and leads to many other theories having assumed directions for the resultant pressure.

**Algebraic Method.**—In Fig. 4, the wall with a height $h$, slopes toward the earth, being inclined to the horizontal at an angle $\theta$, and the earth has a surcharge with slope $\delta$, which is not greater than $\phi$, the angle of repose. It is required to find the pressure $P$ against the retaining wall, it being assumed that the resultant pressure makes an angle $z$ with the back of the wall.

It is assumed that the triangular prism of earth above some plane, the trace of which is the line *AE*, will produce the maximum pressure on the wall and on the earth below the plane, and that in turn the prism will be supported by the reactions of the wall and the earth. Let *OW* represent the weight of the prism *ABE*, the length of the prism being assumed equal to unity, let *OP* be the reaction of the wall, and *OR* be the reaction of the earth below.

Now the forces *OW*, *OP*, and *OR* will be concurrent and will be in equilibrium; *OP* and *OR* will therefore be components of *OW*. When the prism *ABE* is just on the point of moving *OP*

will make an angle with a normal to the back of the wall equal to $z$ (different authorities assume values of $z$ from zero to $\phi'$, the angle of friction of earth on masonry, or $\phi$, the angle of repose of earth); while $OR$ will make an angle with the normal to the plane of rupture $AE$ equal to $\phi$. Let $P$ represent the pressure $OP$ against the wall, $W$ represent the weight of the prism of earth, and $w$ the weight per cu. ft.

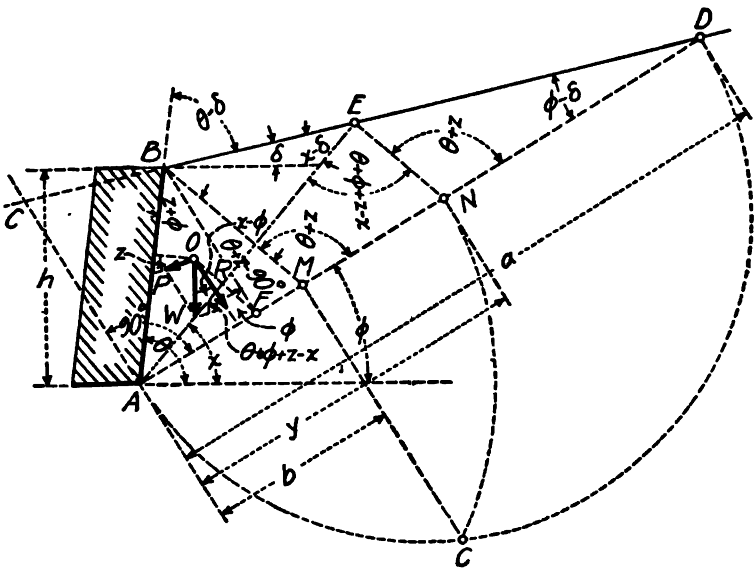


FIG. 4.

In the triangle $OWR$ angle $WOR = x - \phi$, and angle $ORW = \theta + \phi + z - x$. Through $E$ draw $EN$, making the angle $AEN = \theta + \phi + z - x$ with $AE$. Then the triangle $AEN$ is similar to triangle $ORW$, and

$$\frac{P}{W} = \frac{EN}{AN}, \qquad \text{and} \qquad P = W\frac{EN}{AN}$$

But $W$ equals $w \cdot$*area triangle* $ABE = \frac{1}{2}w \cdot AB \cdot BE \cdot \sin(\theta - \delta)$, and

$$P = \tfrac{1}{2}w \cdot \sin(\theta - \delta)\,\frac{AB \cdot BE \cdot EN}{AN} \tag{4}$$

Now $P$ varies with the angle $x$, and will have a maximum value for some value of $x$, which may be found by differentiating (4) and placing the result equal to zero.

Differentiating and substituting in (4) and reducing we have

$$P = \tfrac{1}{2}w \cdot h^2 \frac{\sin^2(\theta - \phi)}{\sin^2\theta \cdot \sin(\theta + z)\left(1 + \sqrt{\dfrac{\sin(z + \phi)\cdot\sin(\phi - \delta)}{\sin(\theta + z)\cdot\sin(\theta - \delta)}}\right)^2} \tag{5}$$

$$= \tfrac{1}{2}w \cdot h^2 \cdot K \tag{6}$$

which is the general formula for the pressure on a retaining wall.

Now if $z$ in (5) is made equal to $\phi'$, the angle of repose of earth on the wall,

$$P = \tfrac{1}{2}w \cdot h^2 \frac{\sin^2(\theta - \phi)}{\sin^2\theta \cdot \sin(\theta + \phi')\left(1 + \sqrt{\dfrac{\sin(\phi + \phi')\cdot\sin(\phi - \delta)}{\sin(\theta + \phi')\cdot\sin(\theta - \delta)}}\right)^2} \tag{7}$$

which is Cain's formula (20) in another form.

If $z$ in (5) is made equal to $\delta$, and $\theta$ made equal to 90°,

$$P = \tfrac{1}{2}w\cdot h^2 \frac{\cos^2\phi}{\cos\delta\left(1+\sqrt{\dfrac{\sin(\phi+\delta)\cdot\sin(\phi-\delta)}{\cos^2\delta}}\right)^2} \tag{8}$$

which is Rankine's formula (2) in another form.

If $z$ in (5) is made equal to zero,

$$P = \tfrac{1}{2}w\cdot h^2 \frac{\sin^2(\theta-\phi)}{\sin^2\theta\left(1+\sqrt{\dfrac{\sin\phi\cdot\sin(\phi-\delta)}{\sin\theta\cdot\sin(\theta-\delta)}}\right)^2} \tag{9}$$

which gives the normal pressure on a wall.

If $\theta$ in (9) = 90°,

$$P = \tfrac{1}{2}w\cdot h^2 \frac{\cos^2\phi}{\left(1+\sqrt{\dfrac{\sin\phi\cdot\sin(\phi-\delta)}{\cos\delta}}\right)^2} \tag{10}$$

If $\delta$ in (10) = 0°,

$$P = \tfrac{1}{2}w\cdot h^2 \frac{\cos^2\phi}{(1+\sin\phi)^2},$$

$$= \tfrac{1}{2}w\cdot h^2 \tan^2(45^\circ - \tfrac{1}{2}\phi) \tag{11}$$

$$= \tfrac{1}{2}w\cdot h^2 \frac{1-\sin\phi}{1+\sin\phi} \tag{12}$$

which is Rankine's formula (1) for a vertical wall without surcharge.

**Graphic Method.**—If the angle $z$, the angle between the back of the wall and a normal to the wall, is known, the resultant pressure on a wall may be calculated by a graphic method, Fig. 5, based on the "theory of a wedge of maximum thrust." The graphic method will be described—the proof of the method is given in "The Design of Walls, Bins and Grain Elevators."

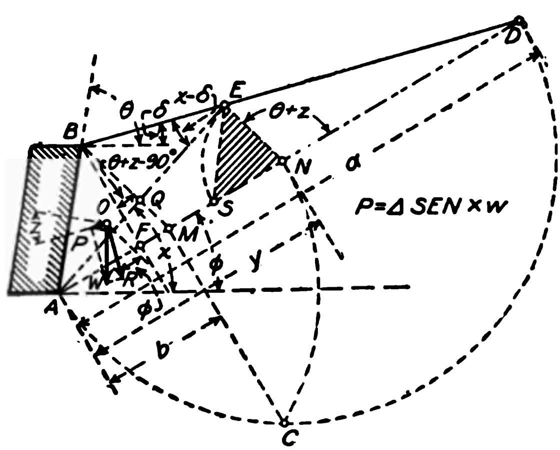



FIG. 5.

In Fig. 5 the retaining wall $AB$ sustains the pressure of the filling with a surcharge $\delta$ and an angle of repose $\phi$. It is required to calculate the resultant pressure $P$.

The graphic solution is as follows: Through $B$ in Fig. 5 draw $BM$ making an angle with $BF$, the normal to $AD$, equal to $\lambda = \theta + z - 90°$, the angle that $P$ makes with the horizontal. With

diameter $AD$ describe arc $ACD$. Draw $MC$ normal to $AD$ and with $A$ as a center and a radius $AC$ describe arc $CN$. Then $AN = y$, $AM = b$ and $y = \sqrt{a \cdot b}$. Draw $EN$ parallel to $BM$. With $N$ as a center and radius $EN$, describe arc $ES$. Then $AE$ is the trace of the plane of rupture, and $P$ = area $SEN \cdot w$.

**Cain's Formulas.***—Professor William Cain assumes that the angle $z$ is equal to $\phi'$, the angle of friction of the filling on the back of the wall. By substituting in (5) we have for a

*Vertical Wall With Level Surface*, $\delta = 0$.

$$P = \tfrac{1}{2} w \cdot h^2 \left(\frac{\cos \phi}{n + 1}\right)^2 \frac{1}{\cos \phi'} \tag{13}$$

where

$$n = \sqrt{\frac{\sin (\phi + \phi') \cdot \sin \phi}{\cos \phi'}}$$

If $\phi = \phi'$, then $n = \sqrt{2} \sin \phi$, and

$$P = \tfrac{1}{2} w \cdot h^2 \frac{\cos \phi}{(1 + \sin \phi \sqrt{2})^2} \tag{14}$$

If $\phi' = 0$, then

$$P = \tfrac{1}{2} w \cdot h^2 \cdot \tan^2 \left(45^\circ - \frac{\phi}{2}\right) \tag{15}$$

*Vertical Wall With Surcharge* $= \delta$.

$$P = \tfrac{1}{2} w \cdot h^2 \left(\frac{\cos \phi}{n + 1}\right)^2 \frac{1}{\cos \phi'} \tag{16}$$

where

$$n = \sqrt{\frac{\sin (\phi + \phi') \cdot \sin (\phi - \delta)}{\cos \phi' \cdot \cos \delta}}$$

If $\delta = \phi$,

$$P = \tfrac{1}{2} w \cdot h^2 \frac{\cos^2 \phi}{\cos \phi'} \tag{17}$$

If $\phi' = 0$, and $\delta = \phi$,

$$P = \tfrac{1}{2} w \cdot h^2 \cdot \cos^2 \phi \tag{18}$$

*Inclined Wall With Horizontal Surface.*

$$P = \tfrac{1}{2} w \cdot h^2 \left(\frac{\sin (\theta - \phi)}{(n + 1) \sin \theta}\right)^2 \frac{1}{\sin (\phi' + \theta)} \tag{19}$$

where

$$n = \sqrt{\frac{\sin (\phi + \phi') \cdot \sin \phi}{\sin (\phi' + \theta) \cdot \sin \theta}}$$

*Inclined Wall With Surcharge* $= \delta$.

$$P = \tfrac{1}{2} w \cdot h^2 \left(\frac{\sin (\theta - \phi)}{(n + 1) \cdot \sin \theta}\right)^2 \frac{1}{\sin (\phi' + \theta)} \tag{20}$$

where

$$n = \sqrt{\frac{\sin (\phi + \phi') \cdot \sin (\phi - \delta)}{\sin (\phi' + \theta) \cdot \sin (\theta - \delta)}}$$

**Wall With Loaded Filling.**—In Fig. 6, the filling is loaded with a uniformly distributed load. Calculate $h_1$ by dividing the loading per sq. ft. by $w$. Let $h + h_1 = H$. Then the resultant pressure for a wall with height $H$, will be

$$P_2 = \tfrac{1}{2} w \cdot H^2 \cdot K \tag{21}$$

and the resultant pressure for a wall with height $h_1$, will be

$$P_1 = \tfrac{1}{2} w \cdot h_1^2 \cdot K \tag{22}$$

* Professor Rebhann makes the same assumptions and uses the graphic method of Fig. 5.

The pressure on the wall $AD$ will be

$$P = P_2 - P_1 = \tfrac{1}{2}w(H^2 - h_1^2)K \tag{23}$$

and the point of application is through the center of gravity of $ADGE$, which makes

$$y_1 = \tfrac{1}{3}\,\frac{H^2 + H\cdot h_1 - 2h_1^2}{H + h_1} \tag{24}$$

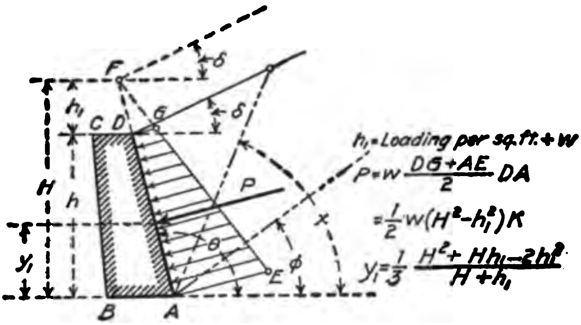



FIG. 6.

**Walls With Negative Surcharge.**—For the calculation of the pressures on retaining walls with negative surcharge, $\delta$ negative, see the author's "The Design of Walls, Bins and Grain Elevators," second edition.

**STABILITY OF RETAINING WALLS.**—A retaining wall must be stable (1) against overturning, (2) against sliding, and (3) against crushing the masonry or the foundation.

The factor of safety of a retaining wall is the ratio of the weight of a filling having the same angle of internal friction that will just cause failure to the actual weight of the filling. For a factor of safety of 2 the wall would just be on the point of failure with a filling weighing twice that for which the wall is built.

1. **Overturning.**—In Fig. 7, let $P$, represented by $OP'$, be the resultant pressure of the earth, and $W$, represented by $OW$, be the weight of the wall acting through its center of gravity. Then $E$, represented by $OR$, will be the resultant pressure tending to overturn the wall.

Draw $OS$ through the point $A$. For this condition the wall will be just on the point of overturning, and the factor of safety against overturning will be unity. The factor of safety for $E = OR$ will be

$$f_0 = SW/RW \tag{25}$$

2. **Sliding.**—In Fig. 7 construct the angle $H1G$ equal to $\phi'$, the angle of friction of the masonry on the foundation. Now if $E$ passes through 1, and takes the direction $OQ$, the wall will be on the point of sliding, and the factor of safety against sliding, $f_s$, will be unity. For $E = OR$, the factor of safety against sliding will be

$$f_s = QM'/RM \tag{26}$$

Retaining walls seldom fail by sliding.

The factor of safety against sliding is sometimes given as

$$f_s = \frac{F}{H}\tan\phi'. \tag{27}$$

where $H$ is the horizontal component of $P$. Equations (26) and (27) give the same values only where the resultant $P$ is horizontal.

3. **Crushing.**—In Fig. 7 the load on the foundation will be due to a vertical force $F$, which produces a uniform stress, $p_1 = F/d$, over the area of the base, and a bending moment $= F\cdot b$, which produces compression, $p_2$, on the front and tension, $p_2$, on the back of the foundation.

The sum of the tensile stresses due to bending must equal the sum of the compressive stresses, $= \frac{1}{4}p_2d$. These stresses act as a couple through the centers of gravity of the stress triangles on each side, and the resisting moment is

$$M' = \frac{1}{4}p_2\cdot d\cdot \frac{2}{3}d = \frac{1}{6}p_2\cdot d^2 \tag{28}$$

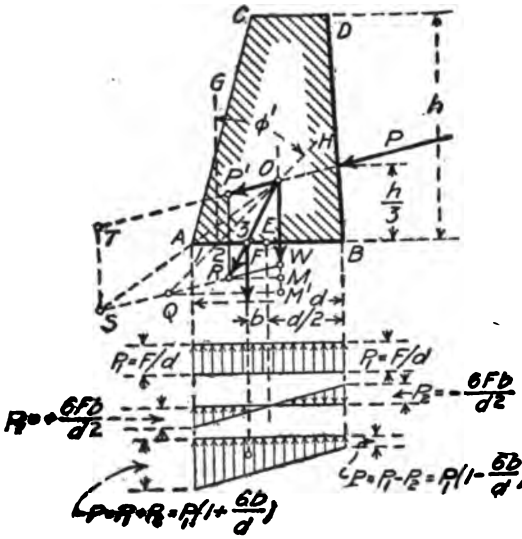



FIG. 7.

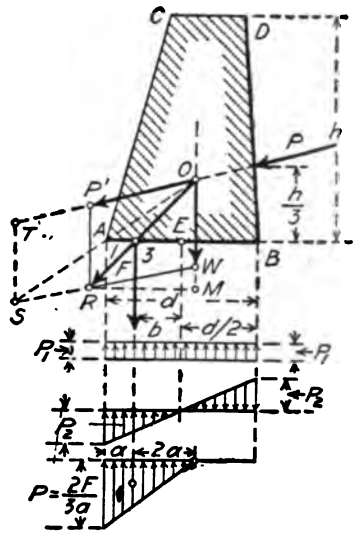



FIG. 8.

But the resisting movement equals the overturning moment, and

$$\frac{1}{6}p_2\cdot d^2 = F\cdot b,$$

and

$$p_2 = \pm\frac{6F\cdot b}{d^2} \tag{29}$$

The total stress on the foundation then is

$$p = p_1 \pm p_2 = p_1(1 \pm 6b/d) \tag{30}$$

Now if $b = \frac{1}{6}d$, we will have

$$p = 2p_1, \quad \text{or} \quad 0.$$

In order therefore that there be no tension, or that the compression never exceed twice the average stress, the resultant should never strike outside the middle third of the base.

If the resultant strikes outside of the middle third of a wall in which the masonry can take no tension, the load will all be taken by compression and can be calculated as follows:

In Fig. 8 the resultant $F$ will pass through the center of gravity of the stress diagram, and will equal the area of the diagram.

$$F = \frac{3}{2}p\cdot a$$

and

$$p = \frac{2F}{3a} \tag{31}$$

which gives a larger value of $p$ than would be given if the masonry could take tension.

**General Principles of Design.**—The overturning moment of a masonry retaining wall of gravity section depends upon the weight of the filling, the angle of internal friction of the filling, the surcharge, and the height and shape of the wall. The resisting moment depends upon the

weight of the masonry, the width of the foundation, and the cross-section of the wall. The most economical section for a masonry retaining wall is obtained when the back slopes toward the filling. In cold localities, however, this form of section may be displaced by heaving due to the action of frost, and it is usual to build retaining walls with a slight batter forwards. The front of the wall is usually built with a batter of from ½ in. to 1 in. in 12 in. In order to keep the center of gravity of the wall back of the center of the base it is necessary to increase the width of the wall at the base by adding a projection to the front side. Where the wall is built on the line of a right of way it is sometimes necessary to increase the width of the base by putting the projection on the rear side, making an L-shaped wall. The weight of the filling upon the base and back of the wall adds to the stability of the wall. Where the wall is built to support an embankment expensive to excavate, it is often economical to make the wall L-shaped, with all the projection on the front side.

In calculating the thrust on retaining walls great care must be exercised in selecting the proper values of $w$ and $\phi$, and the conditions of surcharge. It will be seen from the preceding discussion that the value of the thrust increases very rapidly as $\phi$ decreases, and as the surcharge increases. Where the wall is to sustain an embankment carrying a railroad track, buildings, or other loads, a proper allowance must be made for the surcharge.

The filling back of the wall should be deposited and tamped in approximately horizontal layers, or with layers sloping back from the wall; and a layer of sand, gravel or other porous material should be deposited between the filling and the wall, to drain the filling downwards. To insure drainage of the filling, drains should be provided back of the wall and on top of the footing, and "weep-holes" should be provided near the bottom of the wall at frequent intervals to allow the water to pass through the wall. With walls from 15 to 25 ft. high, it is usual to use "weepers" 4 in. in diameter placed from 15 to 20 ft. apart. The "weepers" should be connected with a longitudinal drain in front of the wall. The filling in front of the wall should also be carefully drained.

The permissible point at which the resultant thrust may strike the base of the foundation will depend upon the material upon which the retaining wall rests. When the foundation is solid rock or the wall is on piles driven to a good refusal, the resultant thrust may strike slightly outside the middle third with little danger to the stability of the wall. When the retaining wall, however, rests upon compressible material the resultant thrust should strike at or inside the center of the base. Where the resultant thrust strikes outside of the center of the base, any settlement of the wall will cause the top to tip forward, causing unsightly cracks and local failure in many cases, and total failure where the settlement is excessive. Where extended footings are used it may be necessary to use some reinforcing steel to prevent a crack in the footing in line with the face of the wall.

Plain masonry walls should be built in sections, the length depending upon the height of the wall, the foundation and other conditions.

Under usual conditions the length of the sections should not exceed 40 ft., 30 ft. sections being preferable, and in no case should the length of the section exceed about three times the height. Separate sections should be held in line and in elevation, either by grooves in the masonry or by means of short bars placed at intervals in the cross-section of the wall, fastened rigidly in one section and sliding freely in the other. The back of the expansion joints should be waterproofed with 3 or 4 layers of burlap and coal tar pitch. The burlap should be about 30 in. wide, and the pitch and the burlap should be applied as on tar and gravel roofs. The joints between the sections of a retaining wall on the front side should be from ⅛ to ¼ of an in. in width, and should be formed by a V-shaped groove made of sheet steel and fastened to the forms while the concrete is being placed. Where there is danger of the water in the filling percolating through the wall or in an alkali country, the surface of the back of the wall should be coated with a waterproof coating. The most satisfactory waterproof coating known to the author is a coal tar paint made by mixing refined coal tar, Portland cement and kerosene in the proportions of 16 parts refined coal tar, 4 parts of Portland cement and 3 parts of kerosene oil. The Portland

cement and kerosene should be mixed thoroughly and the coal tar then added. In cold weather the coal tar may be heated and additional kerosene added to take account of the evaporation. This paint not only covers the surface but combines with it, so that two or three coats are sometimes required. While the surface of the concrete should be dry, coal tar paint will adhere to moist or wet concrete. In building retaining walls in sections, the end of the finished section should be coated with coal tar paint to prevent the adhesion to the next section.

For methods of waterproofing masonry, see methods of waterproofing bridge floors in Chapter XV.

**DESIGN OF RETAINING WALLS.**—The design of masonry retaining walls will be illustrated by the design of the retaining walls for West Alameda Avenue Subway, taken from the author's "The Design of Walls, Bins and Grain Elevators," second edition.

**Design of Retaining Walls for West Alameda Avenue Subway, Denver, Colorado.**—The height of the walls varied from 8 ft. to 29 ft. 3 in., while the foundation soil varied from a compact gravel to a mushy clay. The design of the maximum section, which rests on a compact gravel, will be given. The concrete was mixed in the proportion of 1 part Portland cement, 3 parts sand and 5 parts screened gravel. Crocker and Ketchum, Denver, Colo., were the consulting engineers. The wall is shown in Fig. 9 and in Fig. 10.

The following assumptions were made: Weight of concrete, 150 lb. per cu. ft.; weight of filling, $w$ = 100 lb. per cu. ft.; angle of repose of filling, $1\frac{1}{2} : 1$ ($\phi = 33° 40'$); surcharge, 600 lb. per sq. ft., equivalent to 6 ft. of filling; maximum load on foundation, 6,000 lb. per sq. ft.

**Solution.**—After several trials the following dimensions were taken: Width of coping 2 ft. 6 in., thickness of coping 1 ft. 6 in., batter of face of wall $\frac{1}{2}$ in. in 12 in., batter of back of wall $3\frac{1}{2}$ in. in 12 in., width of base 15 ft. $2\frac{1}{2}$ in. (ratio of base to height = 0.52), front projection of base 4 ft., other dimensions as shown in Fig. 9. The calculations were made for a section of the wall one foot in length.

The property back of the wall will probably be used for the storage of coal, etc., and it was assumed that the surcharge came even with the back edge of the footing of the wall. The resultant pressure of the filling on the plane $A$–2 was calculated by the graphic method of Fig. 5 and Fig. 6, and was found to be $P'$ = 17,290 lb. The weight of the filling in the wedge back of the wall is $W'$ = 16,435 lb., acting through the center of gravity of the filling. The resultant of $P'$ and $W'$ is $P$ = 23,850 lb. = the resultant pressure of the filling on the back of the wall. The weight of the masonry is $W$ = 33,144 lb., acting through the center of gravity of the wall, and the resultant of $P$ and $W$ is $E$ = 52,510 lb. = the resultant pressure of the wall and the filling upon the foundation. The vertical component of $E$ is $F$ = 49,580 lb., and cuts the foundation, $b$ = 2.1 ft. from the middle.

1. *Stability Against Overturning.*—The line $OD$ in this case is nearly parallel to the line $QW$ which brings the point $S$ in Fig. 9 at a great distance from the point $W$. The factor of safety against overturning was calculated on the original drawing and found to be $f_o > 25$.

2. *Stability Against Sliding.*—The coefficient of friction of the masonry on the footing will be assumed to be $\tan \phi' = 0.57$ and $\phi' = 30°$. Through $O$, Fig. 9, draw $OQ$, cutting the base of wall $5A$ at 6, and making an angle $\phi' = 30°$ with a vertical line through 6. Then the factor of safety against sliding will be

$$f_s = QM'/RM = 2.5$$

This is ample as the resistance of the filling in front of the toe will increase the resistance against sliding.

3. *Stability Against Crushing.*—In Fig. 9 the direct pressure will be $p_1 = 49{,}580/15.21 = 3{,}220$ lb. per sq. ft.

The pressure due to bending will be

$p_2 = \pm 6F \cdot b/d^2 = \pm (6 \times 49{,}580 \times 2.1)/231.4 = \pm 2{,}700$ lb. per sq. ft., and the maximum pressure is

$$p = 3{,}220 + 2{,}700 = + 5{,}920 \text{ lb. per sq. ft.}$$

and the minimum pressure is

$$p = 3,220 - 2,700 = + 520 \text{ lb. per sq. ft.}$$

The allowable pressure was 6,000 lb. per sq. ft., so that the pressure is safe for a compact gravel. Where the walls were supported on the mushy clay it was necessary to extend the projection of the footing on the front side and to bring the resultant $F$ to the center of the wall.

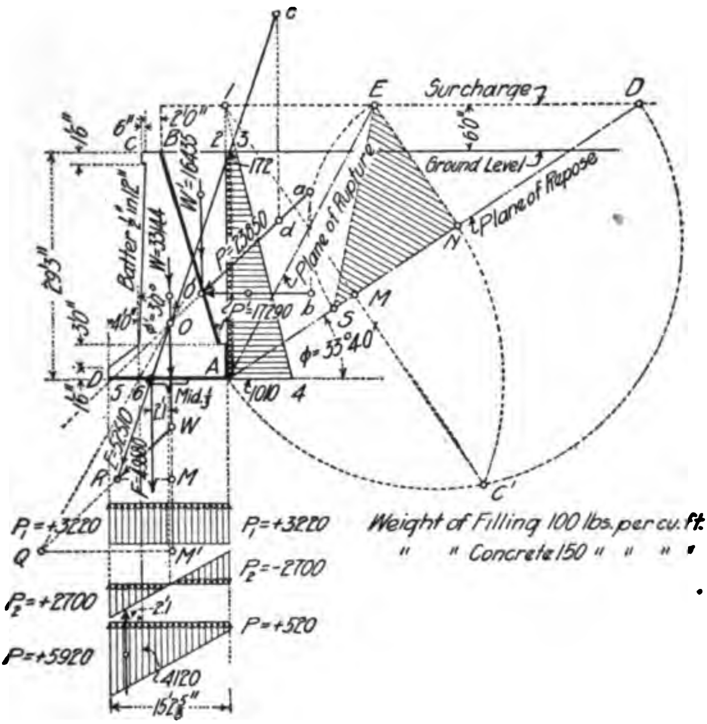



FIG. 9. RETAINING WALL, WEST ALAMEDA AVENUE SUBWAY.

4. *Upward Pressure on Front Projection of Foundation.*—Where projections are used on the foundations of retaining walls it may be necessary to reinforce the base to prevent the projection breaking off in line with the face of the wall. The bending moment of the upward pressure about the front face of the wall from Fig. 9 is

$$M = \tfrac{1}{2}(5,920 + 4,120) \times 4 \times 2.1 \times 12$$
$$= 506,000 \text{ in-lb.}$$

The tension on the concrete at the bottom of the footing will be

$$f = M \cdot c/I = M \cdot d/2I = (506,000 \times 27)/157,464$$
$$= 88 \text{ lb. per sq. in.}$$

Since the ultimate strength of the concrete in tension is approximately 200 lb. per sq. in.,

no reinforcing is required. However, $\frac{3}{4}$ in. □ bars were placed 18 in. centers and 3 in. from the bottom of the foundation.

**Data.**—The coefficients of friction of various materials are given in Table I. The angles of repose of different materials are given in Table II. The conditions of surface and amount of moisture cause wide variations in the coefficients. Additional data for the design of retaining walls are given in Tables III to VI.

### TABLE I.

#### COEFFICIENTS OF FRICTION.

| Materials. | Coefficients. | Materials. | Coefficients. |
|---|---|---|---|
| Dry masonry on dry masonry | 0.6 to 0.7 | Masonry on dry clay | 0.5 to 0.6 |
| Masonry on masonry with wet mortar | 0.75 | Masonry on moist clay | 0.33 |
| Timber on stone | 0.4 | Earth on earth | 0.25 to 1.0 |
| Iron on stone | 0.3 to 0.7 | Hard brick on hard brick | 0.7 |
| Timber on timber | 0.2 to 0.5 | Concrete blocks on concrete blocks | 0.65 |

### TABLE II.

#### ANGLES OF REPOSE, $\phi$, FOR MATERIALS.

| Materials. | $\phi$ | Materials. | $\phi$ |
|---|---|---|---|
| Earth, loam | 30° to 45° | Clay | 25° to 45° |
| Sand, dry | 25° to 35° | Gravel | 30° to 40° |
| Sand, moist | 30° to 45° | Cinders | 25° to 40° |
| Sand, wet | 15° to 30° | Coke | 30° to 45° |

### TABLE III.

#### ALLOWABLE PRESSURE ON FOUNDATIONS.

| Material. | Pressure in Tons per Sq. Ft. |
|---|---|
| Soft clay | 1 to 2 |
| Ordinary clay and dry sand mixed with clay | 2 to 3 |
| Dry sand and clay | 3 to 4 |
| Hard clay and firm, coarse sand | 4 to 6 |
| Firm, coarse sand and gravel | 6 to 8 |
| Bed rock | 15 and up. |

### TABLE IV.

#### ALLOWABLE PRESSURE ON MASONRY.

| Materials. | Pressure in Tons per Sq. Ft. |
|---|---|
| Common brick, Portland cement mortar | 12 |
| Paving brick, Portland cement mortar | 15 |
| Rubble masonry, Portland cement mortar | 12 |
| Sandstone, first class masonry | 20 |
| Limestone, first class masonry | 25 |
| Granite, first class masonry | 30 |
| Portland cement concrete, 1–2–4 | 25 |
| Portland cement concrete, 1–3–6 | 20 |

## TABLE V.

### Weight, Specific Gravity and Crushing Strength of Masonry.

| Materials. | Weight in Pounds per Cubic Foot. | Specific Gravity. | Crushing Strength in Pounds per Square Inch. |
|---|---|---|---|
| Sandstone | 150 | 2.4 | 4,000 to 15,000 |
| Limestone | 160 | 2.6 | 6,000 to 20,000 |
| Trap | 180 | 2.9 | 19,000 to 33,000 |
| Marble | 165 | 2.7 | 8,000 to 20,000 |
| Granite | 165 | 2.7 | 8,000 to 20,000 |
| Paving brick, Portland cement | 150 | 2.4 | 2,000 to 6,000 |
| Stone concrete, Portland cement | 140 to 150 | 2.2 to 2.4 | 2,500 to 4,000 |
| Cinder concrete, Portland cement | 112 | 1.8 | 1,000 to 2,500 |

## TABLE VI.

### Weight of Different Materials.

| Materials. | Wt. per Cu. Ft., Lb. | Materials. | Wt. per Cu. Ft., Lb. |
|---|---|---|---|
| Loam, loose | 75 to 90 | Sand, wet | 110 to 120 |
| Loam, rammed | 90 to 100 | Gravel | 120 to 135 |
| Sand, dry | 90 to 110 | Soft flowing mud | 105 to 120 |

For specifications for concrete, plain and reinforced, see Appendix II.

**Examples of Retaining Walls.**—Details of six masonry retaining walls with a gravity section are given in Fig. 10. These retaining walls represent the best practice. Details of four reinforced concrete retaining walls are given in Fig. 11. For additional examples see the author's "The Design of Walls, Bins and Grain Elevators."

**DESIGN OF RETAINING WALLS AND ABUTMENTS.***—The Committee believes that the intelligent use of theoretical formulas leads to economical and proper design, and therefore recommends that Rankine's formulas which consider that the filling is a granular mass of indefinite extent, without cohesion, be used in the design of retaining walls. It is recommended that retaining walls be designed (*a*) for a level surcharge, or (*b*) for a sloping surcharge at the angle of repose, or (*c*) for a level surcharge with a uniform surcharge loading. Formulas based on Rankine are given for vertical walls, walls leaning away from the filling, and for walls leaning toward the filling.

The use of a fixed ratio of width to height leads to a neglect of the distribution of the pressure on the foundation. This is a question of great importance, since it is well established that movements from the original alignment, due to unequal settlement, form a defect more common than any other. The Committee feels that attention should be called to the importance of making a study of each case in designing a wall, particularly of the weight and character of the filling, and the amount and distribution of the pressure on the bed of foundations.

**DESIGN OF RETAINING WALLS.**—The following nomenclature is recommended:

$\phi$ = the angle of repose of the filling.

$\theta$ = the angle between the back of the wall and a horizontal line passing through the heel of the wall and extending from the back into the fill.

$\delta$ = angle of surcharge, the angle between a horizontal line and the surface of the filling. (It is recommended that values of $\delta = 0$ or $\delta = \phi$ be used.)

$\lambda$ = the angle between the resultant thrust, $P$, and a horizontal line.

$h$ = vertical height of the wall in feet.

$h'$ = height of surcharge in feet.

* Report of the masonry committee of American Railway Engineering Association, adopted March 22, 1917.

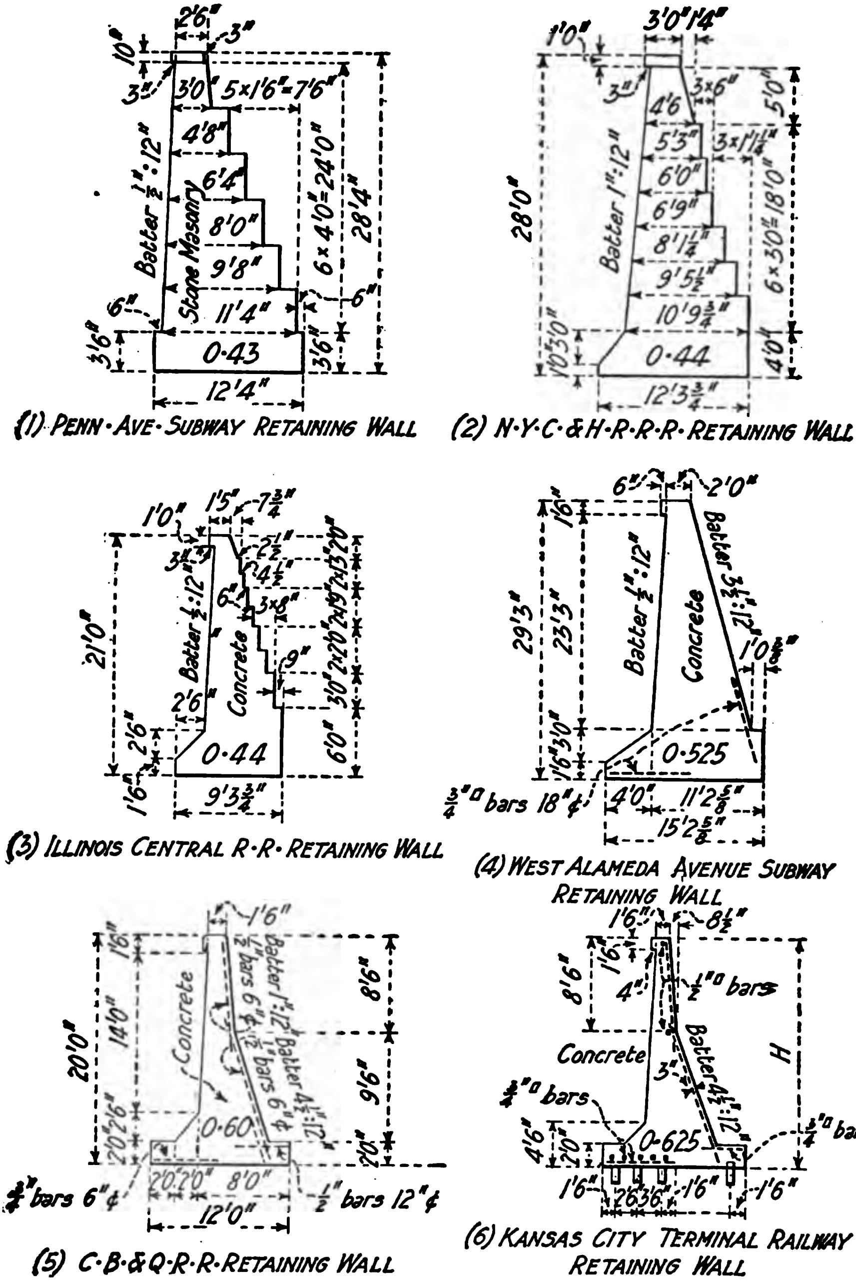



Fig. 10. Examples of Masonry Retaining Walls.

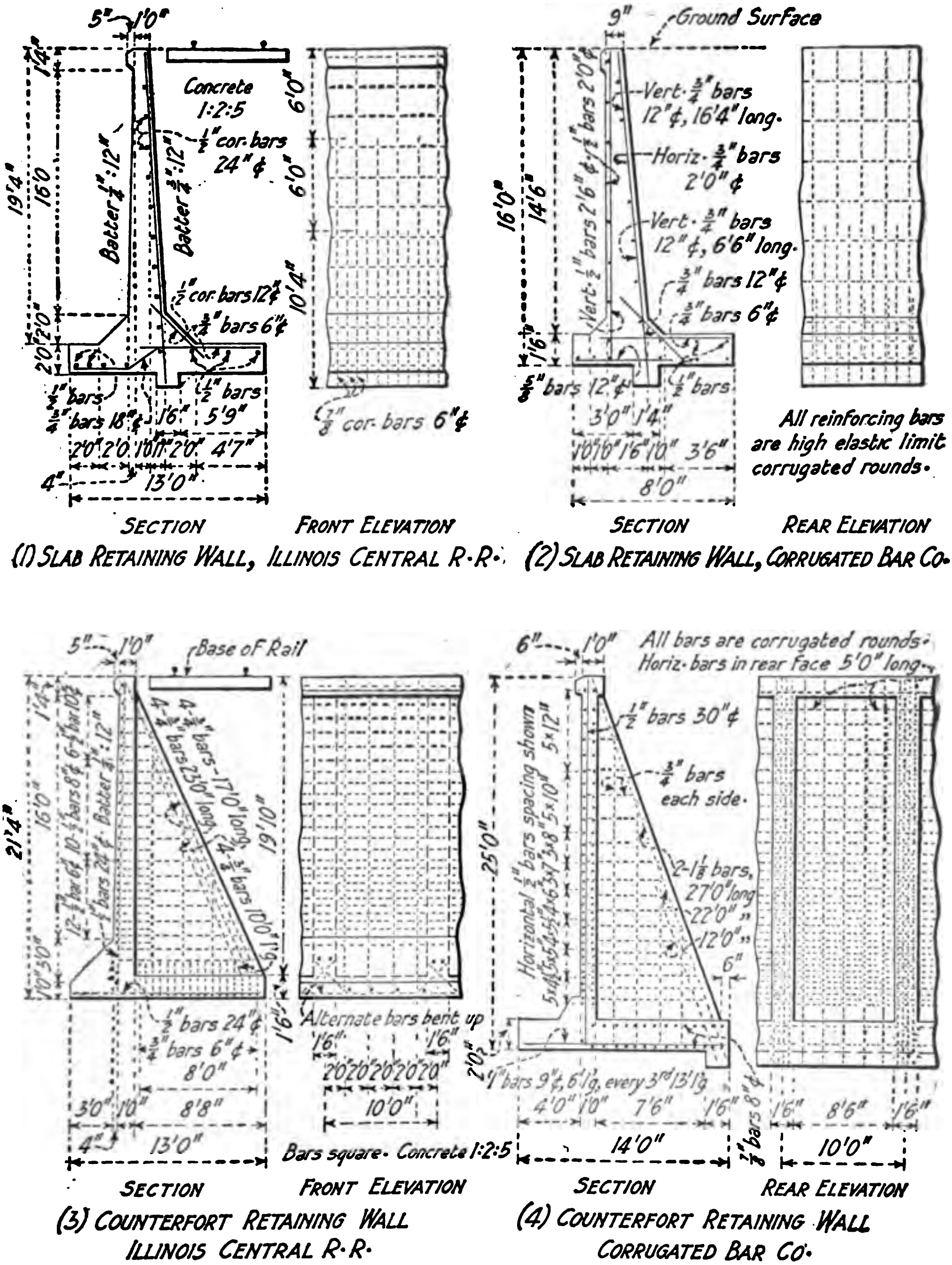



FIG. 11. EXAMPLES OF REINFORCED CONCRETE RETAINING WALLS.

$l$ = width of the base of the wall in feet.
$e$ = distance from the center of the base to the intersection of the resultant thrust, $E$, and the base.
$a = l/2 - e$ = distance from toe of wall to intersection of the resultant thrust, $E$, and the base.
$P$ = the resultant earth pressure per foot of length of wall.
$E$ = the resultant of the earth pressure and the weight of the wall.
$F$ = vertical component of resultant $E$.
$w$ = the weight of the filling per cubic foot.
$w_1$ = the weight of the masonry per foot of length.
$W$ = total weight of the wall per foot of length.
$p_1$ and $p_2$ = pressure per square foot on the foundation, due to $F$, at toe and heel, respectively.

**Formulas.**—The following formulas for vertical walls or for walls leaning away from the filling are based on Rankine's Theory, as given in Howe's "Retaining Walls," and in Ketchum's "Walls, Bins and Grain Elevators"; and the formulas for walls leaning toward the filling are based on a modification of Rankine's Theory, as given in Ketchum's "Walls, Bins and Grain Elevators."

For vertical walls with horizontal surcharge the pressure, $P$, is given by the formula

$$P = \tfrac{1}{2}w\cdot h^2\frac{1 - \sin\phi}{1 + \sin\phi} = \tfrac{1}{2}w\cdot h^2\cdot\tan^2\left(45^\circ - \frac{\phi}{2}\right), \tag{32}$$

where $P$ is parallel to the top surface, is normal to the wall, and is applied at one-third the height of the wall above the base.

For vertical walls with a positive surcharge, $\delta$, the pressure, $P$, is given by the formula

$$P = \tfrac{1}{2}w\cdot h^2\cdot\cos\delta\,\frac{\cos\delta - \sqrt{\cos^2\delta - \cos^2\phi}}{\cos\delta + \sqrt{\cos^2\delta - \cos^2\phi}} \tag{33}$$

where $P$ is parallel to the top surface of the filling, makes an angle $\delta$ with a normal to the back of the wall, and is applied at one-third the height of the wall above the base. Where the surcharge is equal to the angle of repose, $\phi$, formula (33) becomes

$$P = \tfrac{1}{2}w\cdot h^2\cdot\cos\phi \tag{34}$$

For a vertical wall with a loaded surcharge the resultant pressure on the back of the wall will be given by the formula

$$P = \tfrac{1}{2}w\cdot h(h + 2h')\frac{1 - \sin\phi}{1 + \sin\phi} \tag{35}$$

where $h$ is the height of the wall and $h'$ the equivalent height of surcharge, equals surcharge per square foot divided by $w$, the weight per cubic foot of the filling.

The resultant pressure is horizontal and is applied at a distance from the base of the wall equal to

$$y = \frac{h^2 + 3h\cdot h'}{3(h + 2h')} \tag{36}$$

(*a*) In calculating the surcharge due to a track the entire load shall be taken as distributed uniformly over a width of 14 feet for a single track or tracks spaced more than 14 feet centers, and the distance center to center of tracks where tracks are spaced less than 14 feet.

(*b*) In calculating the pressure on a retaining wall where the filling carries permanent tracks or structures, the full effect of the loaded surcharge shall be considered where the edge of the distributed load or the structure is vertically above the back edge of the heel of the wall. The effect of the loaded surcharge may be neglected where the edge of the distributed load or the structure is at a distance from the vertical line through the back edge of the heel of the wall equal to $h$,

Cases 4 to 6 are for walls with heels. The wall may be vertical or may lean forward, or may lean backward, as long as the upper edge of the back of the wall is in front of the vertical plane through the edge of the heel.

Cases 7 to 9 are for walls without heels. Walls with heels come under cases 4 to 6 as long as the upper edge of the back of the wall is in front of the vertical plane through the edge of the heel; if the upper edge of the back of the wall extends back of the vertical plane through the edge of the heel, the problem can be solved by combining the solutions of cases 4 to 6 and 7 to 9.

**Pressure on Foundations.**—The pressures on foundations will be calculated by the following formulas:

Where $a$ is equal to or greater than $l/3$.

Pressure at the toe

$$p_1 = (4l - 6a)\frac{F}{l^2} \tag{37}$$

Pressure at the heel is

$$p_2 = (6a - 2l)\frac{F}{l^2} \tag{38}$$

Where $a$ is less than $l/3$, the pressure at the toe is

$$p_1 = \frac{2F}{3a} \tag{39}$$

**Principles for Design of Retaining Walls.**—The following principles should be observed in the design and construction of retaining walls.

1. For usual conditions of the filling use an angle of repose of $1\frac{1}{2}$ to 1 ($\phi = 33° 42'$). For dry sand or similar material, a slope of 1 to 1 ($\phi = 45°$) may be used.

2. The maximum pressure at the toe of the retaining wall should never exceed the safe bearing pressure on the material considered.

3. When the retaining wall rests on a compressible material, where settlement may be expected, the resultant thrust, $E$, should strike at the middle or back of the middle of the base of the wall so that the wall will settle toward the filling ($a$ = or $> l/2$).

4. When the retaining wall rests on a material where settlement may not be expected the resultant thrust, $E$, should not strike outside the middle third of the base ($a$ = or $> l/3$), except as noted in (5) below.

5. Where the retaining wall rests on solid rock or is carried on piles the resultant thrust, $E$, may strike slightly outside the middle third, provided the wall is safe against overturning, and also provided the maximum allowable pressure is not exceeded.

6. In order that the retaining wall may be safe against sliding, the frictional resistance of the base, combined with the abutting resistance of the earth in front of the wall, must be greater than the horizontal thrust on the back of the wall.

7. The filling back of the wall should be carefully drained so that the wall may not be subjected to hydrostatic pressure.

8. The foundation for a retaining wall should always be placed below frost line.

9. A careful study should be made of the conditions in the design of each wall, and it should be remembered that no theoretical formulas can be more than an aid to the judgment of the experienced designer. The main value of theoretical formulas is in obtaining economical proportions, in obtaining a proper distribution of the stresses, and in making experience already gained more valuable.

## Problems in Design of Retaining Walls.

### PROBLEM 1. INVESTIGATION OF MASONRY WALL.

**Problem.**—Given a plain masonry retaining wall with dimensions as shown in figure in Problem 1 and a surcharge at an angle of 15° 00′. Find the magnitude and direction of the pressure against the wall, the unit pressure at the heel and toe, and the factors of safety against

sliding and overturning. Use Rankine's Method in finding the resultant pressure against the wall and check by Coulomb's Method. The weight of earth is to be assumed as 100 lb. per cu. ft., the angle of repose and the angle of friction 30° 00′, and the angle of surcharge 15° 00′.

**Solution.**—To find the pressure against the wall by Rankine's Method proceed as follows: Draw $AO$ parallel to the surface of the ground and at any convenient point $O$ in $AO$ draw $OD$ at right angles to $AO$. Draw $OM$ vertical and locate $M$ by striking the arc $DM$ with $O$ as a center. Draw OC making an angle $\phi = 30°$ with $OD$. At any point $e$ in $OD$ describe an arc tangent to $OC$ and cutting $OM$ at $f$. Draw $ef$. Through $M$ draw $MG$ parallel to $ef$. Bisect the angle $DGM$ and draw $GR'$. To determine the semi-major axis of the ellipse of stress draw $OR$ parallel

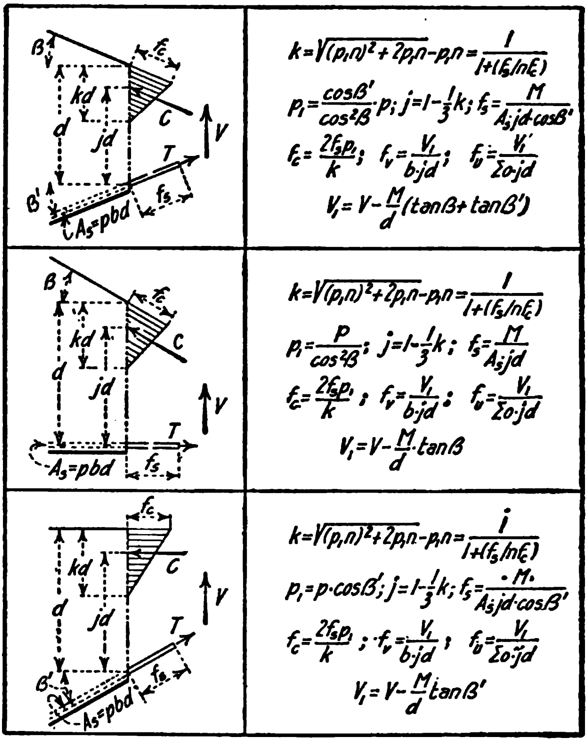
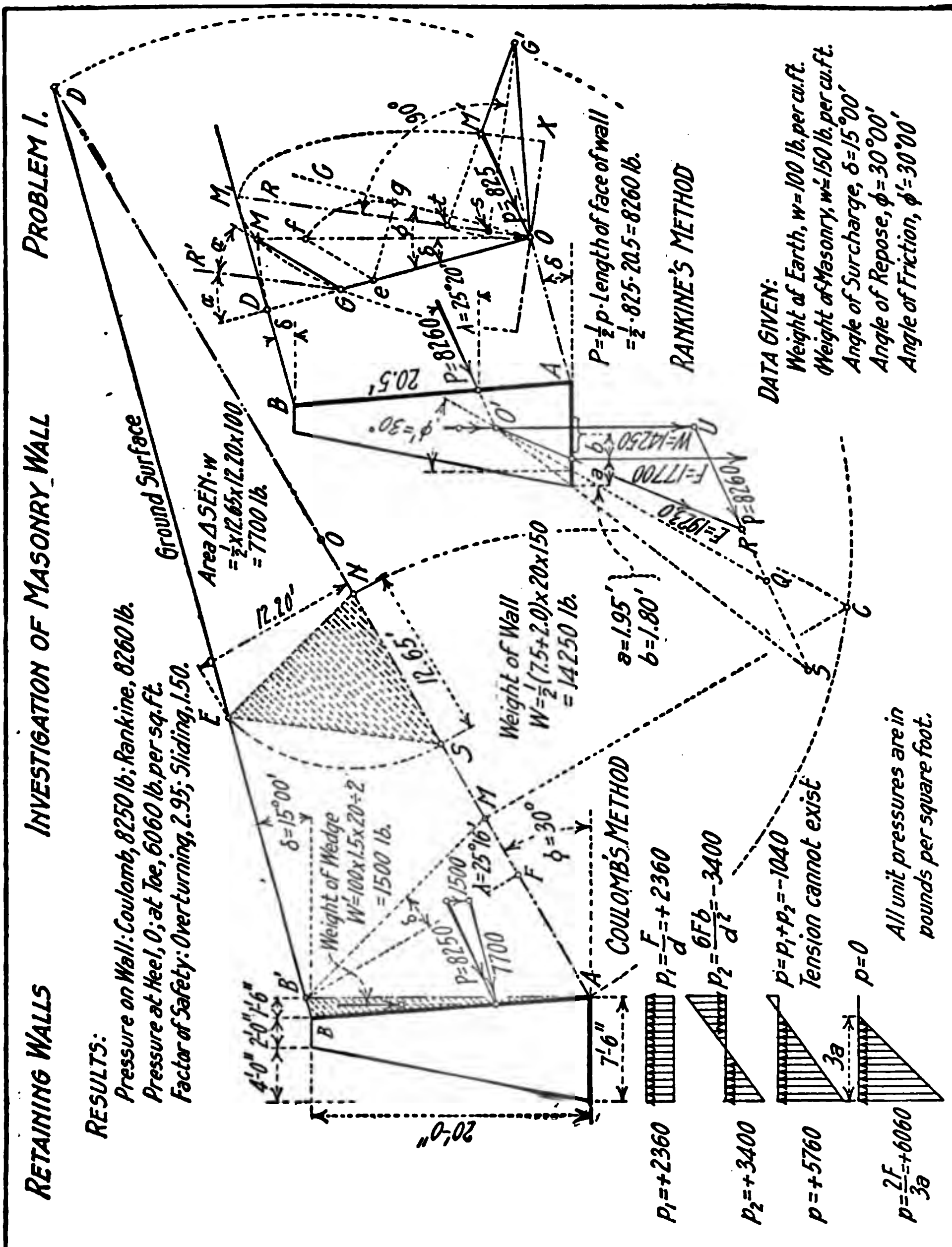



FIG. 14. STRESSES IN WEDGE-SHAPED REINFORCED CONCRETE BEAMS.

to $GR'$ and make $OM_1 = OG + GM$. To determine the semi-minor axis draw OX perpendicular to $OR$ and equal to $OG - GM$. To calculate the unit pressure against the wall at $A$ draw $OG'$ at right angles to the back of the wall $AB$ and make $OG' = OG$, draw $G's$ perpendicular to $OR$ and make $st = Os$, draw $G't$ and lay off $GM'$ equal to $GM$, then $M'O$ acting as shown is the intensity of stress at $A$. To determine the magnitude of this stress $M'O$ measure its length, using the same scale as that used in laying off the wall, and multiply this length by the weight of a cubic foot of earth (100 in this case) and the result will be the intensity of pressure at $A$ measured in pounds per sq. ft. on the surface $AB$. This is found to be $p = 8.25 \times 100 = 825$ lb. per sq. ft. The intensity of pressure at $B$ is evidently zero and since the pressure varies as the depth the total pressure $P$ against the back of the wall $AB$ will be $\frac{1}{2}p \times$ length $AB = \frac{1}{2} \times 825 \times 20.5 = 8,260$ lb. The line of action of $P$ is parallel to $OM'$ and the angle $\lambda$ is measured and found to be 25° 20′.


RETAINING WALLS
INVESTIGATION OF MASONRY WALL
PROBLEM I.
RESULTS:
Pressure on Wall: Coulomb, 8250 lb.; Rankine, 8260 lb.
Pressure at Heel, 0; at Toe, 6060 lb. per sq. ft.
Factor of Safety: Overturning, 2.95; Sliding, 1.50.
Ground Surface
Area ΔSEN·w = ½×12.65×12.20×100 = 7700 lb.
Weight of Wall W=½(7.5+2.0)×20×150 = 14250 lb.
Weight of Wedge W'=100×1.5×20÷2 =1500 lb.
δ=15°00'
λ=25°16'
φ=30°
COULOMB'S METHOD
λ=25°20'
φ'=30°
W=14250
F=17700
F=19730
p=8260
P=½p·Length of face of wall =½·825·20.5=8260 lb.
RANKINE'S METHOD
a=1.95'
b=1.80'
DATA GIVEN:
Weight of Earth, w=100 lb. per cu. ft.
Weight of Masonry, w'=150 lb. per cu. ft.
Angle of Surcharge, δ=15°00'
Angle of Repose, φ=30°00'
Angle of Friction, φ'=30°00'
$p_1=\frac{F}{d}=+2360$
$p_2=\frac{6Fb}{d^2}=-3400$
$p=p_1+p_2=-1040$
Tension cannot exist
$p=0$
All unit pressures are in pounds per square foot.
$p_1=+2360$
$p_2=+3400$
$p=+5760$
$p=\frac{2F}{3a}=+6060$
20'-0"
7'-6"

To find the pressure against the wall by Coulomb's Method proceed as follows: The pressure against the vertical plane $AB'$ must first be found and this pressure combined with the weight of the wedge $ABB'$ to find the pressure against $AB$. This procedure is necessary because Coulomb's method gives the magnitude of the pressure but not its direction, and since it is known that for a vertical plane the pressure is parallel to the ground surface, the magnitude and direction of the resultant pressure can be found by combining the pressure against $AB'$ with the weight of the wedge $ABB'$. Draw $AB'$ vertical and $AD$ making an angle of $\phi = 30°$ with the horizontal, the point $D$ being the intersection of this line, and the ground surface. Through $B'$ draw $BM$ making an angle $\delta = 15°$ with $BF$ the normal to $AD$. Locate $O$ bisecting the line $AD$ and with $O$ as a center and $AO$ as a radius describe the semicircle $ACD$. Draw $MC$ normal to $AD$ and with $A$ as a center and a radius $AC$ describe the arc $CN$. Draw $EN$ parallel to $B'M$. With $N$ as a center and radius $EN$ describe the arc $ES$. Then the total pressure against $AB'$ is $P' =$ area $SEN \cdot w = \frac{1}{2} \times 12.65 \times 12.20 \times 100 = 7{,}700$ lb. acting at $\frac{1}{3}$ the height $AB'$ above $A$. Combine $P'$ with $W'$, the weight of the wedge, acting through its centroid, and find $P = 8{,}250$, the resultant pressure against $AB$. The angle $\lambda$ is measured and found to be $25° 16'$.

These two methods should give the same result. The results obtained in this problem are seen to check very closely.

To find the unit pressure at the heel and toe combine $P = 8{,}260$ with the weight of the wall $W = 14{,}250$, acting through its centroid, giving the resultant $E = 19{,}230$, as shown in the problem under Rankine's Method. This resultant $E$ cuts the base at a distance $b = 1.80'$ from the center, which is outside of the middle third. The unit pressure at the heel and toe are found as shown under Coulomb's Method and are 0 and 6,060 pounds per square foot respectively. The factor of safety against sliding is equal to $QU \div RU = 1.50$, and against overturning is $SU \div RU = 2.95$.

**Results.**—The results of this investigation show that the wall is unsatisfactory, for the resultant pressure on the base falls outside of the middle third. The unit pressures are not excessive if the foundation is dry sand or clay. For proof of construction of ellipse of stress, see the author's Design of Walls, Bins and Grain Elevators.

## PROBLEM 2. INVESTIGATION OF CANTILEVER WALL.

**Problem.**—Make a complete investigation of the reinforced concrete retaining wall with a horizontal surcharge shown in Problem 2. The weight of the wall is 150 lb. per cu. ft. and the earth 100 lb. per cu. ft. The angle of repose is $30° 00'$.

**Solution.**—The first step in the solution is to determine the pressure acting against the back of the wall. This pressure is found by combining the pressure $P'$ against the vertical plane $A'B'$, acting at one-third of the height, with the weight $W'$ of the wedge of earth resting on the heel acting through the center of gravity $G'$ of the wedge. The calculations are shown on the problem.

The resultant pressure on the foundation $R = 22{,}600$ lb. is found by combining the pressure $P$ on the wall with the weight of the wall $W$ acting through the center of gravity $G$. This resultant pressure $R$ cuts the base at a distance of $e = 1.35$ ft. from the center of the base. The unit pressure at the heel and toe are calculated as shown and are 517 and 2,945 lb. per sq. ft. respectively.

The wall is divided into three cantilever beams which must be investigated, i. e., the vertical slab, the heel, and the toe. Three sections, $A$–$A$, $B$–$B$ and $C$–$C$ on the vertical slab must be investigated on account of the change in steel area at these sections. The intensity of stress at $A$–$A$ is $p = w \cdot h \cdot k = 100 \times 6 \times 0.333 = 200$ lb. per sq. ft., where $k = (1 - \sin \phi) \div (1 + \sin \phi)$, $= 0.333$. The total pressure on the cantilever above $A$–$A$ is $P_A = \frac{1}{2} \times 200 \times 6 = 600$ lb. acting 2.0 ft. above $A$–$A$. These values are recorded in the table. The shear at $A$–$A$ is $V = P_A = 600$ lb. The bending moment at $A$–$A$ is $M = 600 \times 2 = 1{,}200$ ft.-lb. $= 14{,}400$ in.-lb. The dimensions, steel area, etc., at this section are as given in the table. The values of $k$ and $j$ may be taken from Fig. 2, Chap. XVIII, or from formulas, $k = \sqrt{p^2 \cdot n^2 + 2p \cdot n} - p \cdot n$ and $j = 1 - \frac{1}{3}k$, where $n = 15$. The unit stresses can now be calculated from the formulas given in the table and are there recorded. The other sections of the vertical slab are investigated in a similar manner.

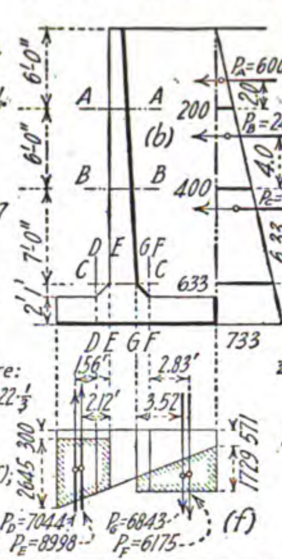
RETAINING WALLS — INVESTIGATION OF CANTILEVER WALL — PROBLEM 2.

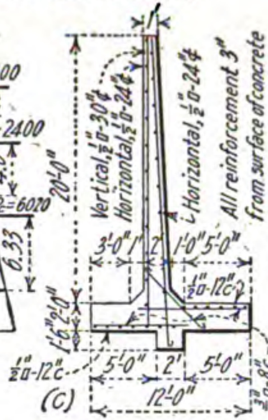
Weight of Wall:
Vertical slab,
$\frac{1}{2}(1+2)\cdot 20\cdot 150=4500$
Base,
$2\cdot 12\cdot 150=3600$
Total, $W=8100$

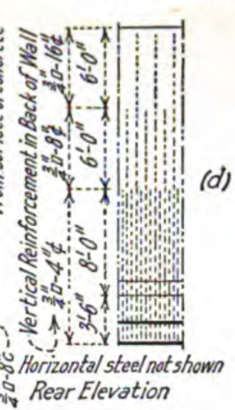
Value of $k$, ($\phi=30°$):
$k=\frac{1-\sin\phi}{1+\sin\phi}=\frac{1-0.5}{1+0.5}$
$=\frac{1}{3}$.


$A'$ $p=733$
Unit Pressure:
$p=whk=100\cdot 22\cdot\frac{1}{3}$
$=733$

Factors of Safety:
Sliding (without cutoff);
$12{,}200\div 8067=1.51$.
Overturning-Safe




Length of embedment to develop 16000 lb/sq.in.
Vertical slab, actual 39", required, 25".
Heel, " 40", " , 37".
Toe, " 40", " , 37".
Diagonal bars, " 33", " , 25".

$p=\frac{V}{d}\pm\frac{6Ve}{d^2}=\frac{21100}{12}\pm\frac{6\times 21100\times 1.35}{12^2}$
$=+2945$ and $+571$
Allowable bearing pressure = 4000.
All forces are in lb. per lin. ft. of wall.
All unit pressures in lb. per sq. ft.

| Section | Force P also V | Arm a | Moment M 12Pa | Depth d | Width b | Steel Size and Spac. | Steel Area A | Steel p $\frac{A}{bd}$ | Steel $\Sigma o$ | Constants k from diagram | Constants j from diagram | Unit Stresses $f_s$ $\frac{M}{Ajd}$ | $f_c$ $\frac{2f_sp}{k}$ | $f_v$ $\frac{V}{bjd}$ | $f_u$ $\frac{f_vb}{\Sigma o}$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | lb. | ft. | in-lb. | in. | in. | in | in.² | | in. | | | pounds per sq. in. | | | |
| A-A | 600 | 2.00 | 14400 | 12.6 | 12 | $\frac{3}{4}$□-16c | .42 | .0028 | 2.25 | .25 | .92 | 2960 | 70 | 4 | 20 |
| B-B | 2400 | 4.00 | 115000 | 16.2 | " | " 8" | .84 | .0043 | 4.50 | .30 | .90 | 9400 | 270 | 14 | 37 |
| C-C | 6020 | 6.33 | 457000 | 20.4 | " | " 4" | 1.69 | .0069 | 9.00 | .36 | .88 | 15100 | 580 | 28 | 37 |
| D-D | 7044 | 1.56 | 132000 | 21.0 | " | " 8" | .84 | .0033 | 4.50 | .27 | .91 | 8220 | 200 | 31 | 83 |
| E-E | 8998 | 2.12 | 229000 | 33.0 | " | " 8" | .84 | ×.0042 | 4.50 | .30 | .90 | 9200 | 260 | ×-3 | -8 |
| F-F | 6175 | 2.83 | 210000 | 21.0 | " | " 8" | .84 | .0033 | 4.50 | .27 | .91 | 13100 | 320 | 27 | 72 |
| G-G | 6843 | 3.52 | 288000 | 32.0 | " | " 8" | .84 | †.0016 | 4.50 | .20 | .93 | 16300 | 260 | ×-6 | -16 |

× See formulas for wedge-shaped beams. † Horiz. steel neglected at G-G.

DATA
Weight of concrete 150 lb. per cu. ft.
Earth, 100 lb. per cu. ft.
Angle of repose, 30°00'
Angle of friction, 30°00'
All. bearing press. 2 T/sq. ft.
Allowable stresses, lb./sq. in.
$f_s=16000$; $f_c=650$
$f_v=40$; $f_u=80$; $n=15$
$f_u=120$ (hooked ends)

The resultant pressure on the toe is obtained by subtracting the downward force due to the weight of the toe from the upward force due to the foundation pressures. This gives a resultant pressure upward as shown graphically by the shaded area in (*f*) on the problem. The shear at the section is given by the portion of the shaded area to the left of the section, and the moment by the moment of this area about the section. These values are recorded in the table. The stresses at *D–D* are found in the same way as explained for *A–A*. On account of the fillet the stresses at the section *E–E* must be found by using the formulas for wedge-shaped beams, Fig. 14.

The resultant pressure on the heel is obtained by subtracting the downward force due to the weight of the heel and the earth on the heel from the upward force due to foundation pressures. This gives a resultant pressure acting downward as shown by the shaded area in (*f*). The unit stresses at *F–F* and *G–G* are found as in a similar manner to that explained for the sections *D–D* and *E–E*. In investigating the section *G–G* the horizontal steel was neglected, for since it is so near the neutral axis it would be carrying very little stress, and the solution considering this steel is quite laborious. The unit stress in steel at this section would be somewhat less than the value of 16,300 lb. per sq. in. given in the table. In investigating the section *F–F* the diagonal steel was neglected, for if the section had been taken a little to the right of its present position the steel would be so near the neutral axis that it would not be effective and the moment would have been reduced but a small amount.

The required length of embedment beyond the section of zero moment is figured for a stress of 16,000 lb. per sq. in. in the steel and a bond stress of 80 lb. per sq. in. if the ends are not hooked, and 120 lb. per sq. in. if hooked.

The factor of safety against overturning is always safe in this type of wall. The factor of safety of sliding, neglecting the cut-off, is equal to the ratio of $V \cdot \tan \phi'$ to $P = 21{,}100 \times 0.577 \div 8{,}067 = 1.51$.

The percentage of temperature reinforcement in the vertical slab is $0.25 \div 16.2 \times 12 = 0.0013$, using the section *B–B* as an average section. This should be at least 0.0033 to conform to the best practice.

**Results.**—The results of the investigation are shown on the problem. The wall is found to be satisfactory in every respect except for the temperature reinforcement.

## PROBLEM 3. DESIGN OF A CANTILEVER RETAINING WALL.

**Problem.**—Design a cantilever wall having a total height of 20 ft. with tracks spaced 14 ft. running parallel to the wall and carrying Cooper's *E* 50 loading. The weight of concrete is 150 lb. per cu. ft. and of earth 100 lb. per cu. ft. The angle of repose and the angle of internal friction are $1\frac{1}{2}$ to 1 (33° 42′). The allowable unit stresses are tension in steel 16,000 lb. per sq. in., compression in concrete 650 lb. per sq. in., allowable bond stress 80 lb. per sq. in., if ends of bars are not hooked, and 120 lb. per sq. in. if ends are hooked, $n = 15$. The allowable pressure on the foundations is 2 tons per sq. ft.

**Solution.**—The effect of the train load may be taken care of by using an equivalent surcharge. The axles are spaced 5 ft. and the axle load is 50,000 lb., and since the tracks are spaced 14 ft., the load per square foot is $50{,}000/(5 \times 14) = 720$ lb. This is equivalent to a surcharge $720 \div 100 = 7.2$ ft. high. A height of 8 ft. will be used. It is not necessary to consider impact in figuring the earth pressure due to engine loads. Two cases must be considered: (1) when the first track is not loaded and there is therefore no live load over the heel, and (2) when all of the tracks are loaded and there is therefore a live load over the heel. The first case is usually the more severe. See first part of this chapter.

The width of base which will make the resultant pressure on the base pass through the outer edge of the middle third was determined for Case 1 from the diagram in (*c*) Fig. 4, page 164*h* in the author's "Design of Walls, Bins and Grain Elevators," for $n = 0.4$ to be $d = 13.7$ ft. For Case 2 from diagram (*d*), for $n = 0.4$, $d = 11.7$ ft. Case 1 gives the maximum width of base. Use $d = 14$ ft.

RETAINING WALLS — DESIGN OF CANTILEVER WALL — PROBLEM 3.

Weight of Wall:
Vertical, $9\cdot(1+2.33)\cdot150=4500$
Base, $2\cdot14\cdot150=\underline{4200}$
Total weight of wall, $W=8700$

Wedge on Heel:
$9\cdot(7+7.83)\cdot100$
$=13350$

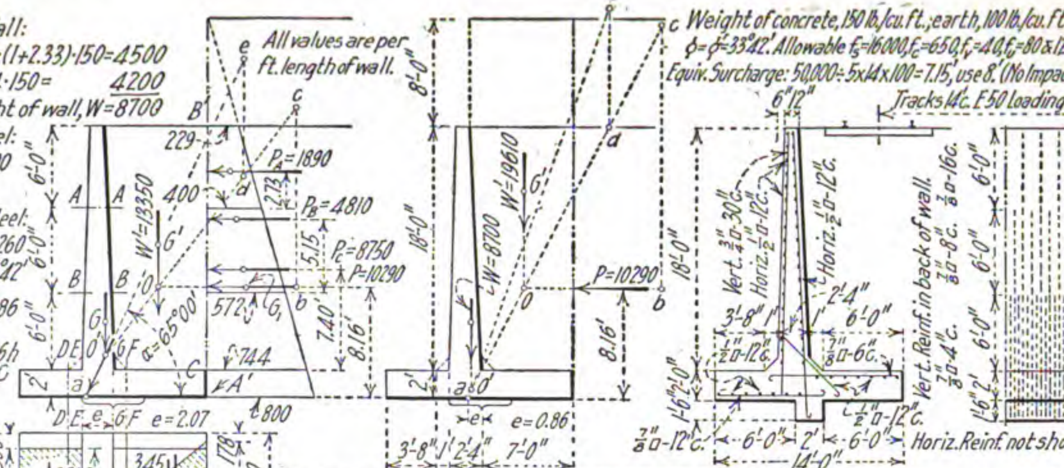
Surcharge on Heel:
$8\cdot7.83\cdot100=6260$

Value of $k$, $\phi=33°42'$

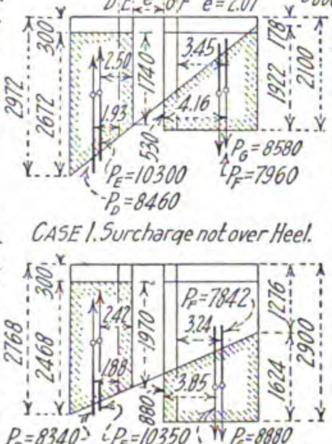
$k=\frac{1-\sin\phi}{1+\sin\phi}=.286$

$p=whk=28.6h$

Weight of concrete, 150 lb./cu.ft.; earth, 100 lb./cu.ft.
$\phi=\phi'=33°42'$. Allowable $f_s=16000$, $f_c=650$, $f_v=40$, $f_u=80$ & $120$.
Equiv. Surcharge: $50,000\div5\times14\times100=7.15'$, use $8'$. (No Impact)




$p=\frac{22050}{14}\left(1\pm\frac{6\cdot2.07}{14}\right)$
$=2972$ and $178$ lb./sq.ft.




CASE 1. Surcharge not over Heel.

$p=\frac{V}{l}\left(1\pm\frac{6e}{l}\right)=\frac{28310}{14}\left(1\pm\frac{6\cdot0.86}{14}\right)$
$=2768$ and $1276$ lb./sq.ft.

CASE 2. Surcharge over Heel.

| Section | Force | Arm | Moment | Net Depth | | | Steel Required | | | Steel Used | | | Constants | | | Unit Stresses | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | $P$ or $V$ | $a$ | $M$ | $d$ | $d$ | $d$ | $R$ | $p$ | $A$ | Size and Spac. | $A$ | $\Sigma o$ | $p$ | $k$ | $j$ | $f_s$ | $f_c$ | $f_v$ | $f_u$ |
| | | | $12Pa$ | $\frac{V}{35b}$ | $\sqrt{\frac{M}{107.5b}}$ | Adopted | $\frac{M}{bd^2}$ | Diagram | $pbd$ | | | | $\frac{A}{bd}$ | From Diagram | | $\frac{M}{Ajd}$ | $\frac{2pf_s}{k}$ | $\frac{V}{bjd}$ | $\frac{f_v b}{\Sigma o}$ |
| | lb. | ft. | in.lb. | in. | in. | in. | | | in.² | in. | in.² | in. | | | | lb. per sq.in. | | | |
| A | 1890 | 2.73 | 61900 | 4.5 | 6.9 | 14.3 | 25.2 | .0017 | 0.29 | $\frac{7}{8}''$□16"c. | 0.57 | 26 | .0033 | .270 | .910 | 8350 | 200 | 12 | 56 |
| B | 4810 | 5.15 | 297000 | 11.5 | 15.2 | 19.6 | 64.4 | .0045 | 1.06 | " 8"c. | 1.15 | 52 | .0049 | .316 | .895 | 14700 | 460 | 23 | 53 |
| C | 8750 | 7.40 | 777000 | 20.8 | 24.5 | 25.0 | 103.6 | .0074 | 2.22 | " 4"c. | 2.30 | 10.5 | .0077 | .378 | .874 | 15500 | 610 | 33 | 38 |
| D | 8460 | 1.93 | 196000 | 20.1 | 12.3 | 21.0 | 37.0 | .0025 | 0.63 | " 12"c. | 0.77 | 3.5 | .0031 | .260 | .910 | 13300 | 320 | 37 | 125 |
| | 8340 | 1.89 | – | – | – | " | – | – | " | " " | " | " | " | " | " | – | – | – | – |
| E | 10300 | 2.50 | 309000 | – | 15.5 | 33.0 | 23.6 | .0015 | 0.59 | " " | " | " | *.0039 | .290 | .900 | 13500 | *360 | – | – |
| | 10350 | 2.42 | 301000 | 24.7 | – | " | – | – | " | " " | " | " | " | " | " | – | – | *3 | *10 |
| F | 7960 | 3.45 | 330000 | 19.0 | 16.0 | 21.0 | 62.4 | .0044 | 1.11 | " 6"c. | 1.53 | 7.0 | .0061 | .345 | .885 | 11600 | 410 | 36 | 61 |
| | 7842 | 3.24 | – | – | – | " | – | – | – | " " | " | " | " | " | " | – | – | – | – |
| †G | 8580 | 4.16 | 428000 | – | 18.2 | 21.0 | 80.8 | .0058 | 1.46 | " 6"c. | 1.53 | 7.0 | .0061 | .345 | .885 | 15000 | 530 | – | – |
| | 8880 | 3.85 | 410000 | 21.1 | – | " | – | – | " | " " | " | " | " | " | " | – | – | 40 | 69 |

* See formulas for wedge-shaped beams. † Fillet neglected at Section 6-6.

The minimum top width which should be used is 12 in. and for a wall of this size the base slab should be about 2 ft. thick. In some cases it may be more economical to use a thickness of 12 in. at the heel and toe and taper the base slab up to the required thickness at the junction of the base and the vertical slab.

In designing this type of wall the section may be considered as divided into three cantilever beams, i. e., the vertical slab, the heel, and the toe. The first step is to determine the thickness of the vertical slab where it joins the base. This section will be called $C$–$C$. The effect of the fillet will be neglected for this section. The horizontal pressure on the vertical slab is the same as on the portion of the plane $A'B$ between $C$–$C$ and the top of the wall. The unit pressure at a depth of $18 + 8 = 26$ ft. from the top of the surcharge is $p = w \cdot h \cdot k = 100 \times 26 \times 0.286 = 744$ lb. per sq. ft. The unit pressure at the top of the wall or 8 ft. below the top of the surcharge is $p = 100 \times 8 \times 0.286 = 229$. The total pressure above the section $C$–$C$ is $P_c = \frac{1}{2}(229 + 744) \times 18 = 8{,}750$ lb. per foot of length of wall. The distance from $C$–$C$ to the line of action of $P_c$ may be found graphically or calculated from the formula

$$y = \frac{2a + b}{a + b} \cdot \frac{h}{3} = \frac{2 \times 229 + 744}{229 + 744} \cdot \frac{18}{3} = 7.40 \text{ ft.}$$

where $a$ and $b$ are the bases of the trapezoid and $h$ the height. The values of $P_c$ and $y$ are recorded in the table. The shear at the section is $V = P_c = 8{,}750$ lb. and the moment is $M = 8{,}750 \times 7.40 = 64{,}800$ ft.-lb. $= 777{,}000$ in.-lb. Using an allowable unit shear of 40 lb. per sq. in., which corresponds to an average shear of 35 lb. per sq. in., the thickness required by shear is $d = V \div 35b = 8{,}750 \div 35 \times 12 = 20.8$ in. The coefficient of resistance for $f_s = 16{,}000$ and $f_c = 650$ is 107.5; the depth required for a moment of 777,000 in.-lb. is therefore (see formula (6c), Chap. XVIII).

$$d = \sqrt{777{,}000 \div 107.5 \times 12} = 24.5 \text{ in.}$$

A depth of 25 in. will be adopted with 3 in. of concrete outside of the steel, making a total depth of 28 in. The dimensions of the vertical slab are now known, the batter of the face being taken as 6 in. in 18 ft. The front of the vertical slab is placed at a distance of $\frac{2}{3} \times 14 = 9.33$ ft. from $A'$, or 4 ft. 8 in. from the toe.

The foundation pressures at the heel and toe will now be found. The unit horizontal pressure at $A'$ is $p = w \cdot h \cdot k = 100 \times 28 \times 0.286 = 800$ lb. per sq. ft. The total pressure against $A'B'$ is $P = \frac{1}{2}(229 + 800) \times 20 = 10{,}290$ lb. for both cases. The distance of $P$ from the base is found to be 8.16 ft. as explained for $P_c$. It is evident that there can be no horizontal pressure acting above the top of the wall for there is nothing for it to act against. In finding the resultant pressure on the wall the horizontal pressure $P$ is combined with the weight over the heel $W'$ acting through its centroid. This pressure on the wall is combined with the weight of the wall acting through its centroid. This resultant pressure cuts the base at a distance from the center of $e = 2.07$ ft. for Case 1 and 0.86 ft. for Case 2. The unit pressures at the toe and heel are now calculated as shown on the problem. These are all within the allowable pressure and the resultant pressure on the foundations strikes within the middle third so the width of base is satisfactory.

The resultant pressure on the toe is found by subtracting the downward force due to the weight of the toe from the upward force due to the foundation pressures. This pressure is shown by the shaded area in the foundation pressure diagram. The shear at the section $D$–$D$ is equal to the portion of this area to the left of the section, and the bending moment is equal to the moment of this portion of the area about the section. The values of the shear and moment for these two cases are recorded in the table, and the depth, steel area, and unit stresses are worked out as explained for the section $C$–$C$. Section $E$–$E$ is worked out in a similar manner but it is found that a fillet is required so the formulas for wedge-shaped beams must be used.

The resultant pressure on the heel is found by subtracting the downward force due to the weight of the heel and the load on the heel, from the upward force due to foundation pressures. This gives a resultant force downward as shown by the shaded areas in the foundation pressure diagrams. The shear and moment at the sections $F$–$F$ and $G$–$G$ were calculated as explained for

*D–D*. A fillet is not necessary but one will be put in to correspond with that on the front of the wall. The reinforcement in this fillet will be made nominal and the effect of the fillet will be neglected in the calculations, assuming all of the tension to be carried by the horizontal steel. The diagonal steel in the fillet will carry a high unit stress, but this will have no effect on the strength of the wall.

The calculations for all sections of the wall are given in the table. The minimum factor of safety against sliding is for Case 1 where the resultant pressure on the foundation makes an angle of 65° with the horizontal, giving a factor of safety of $\tan 33° 42' \div \tan (90° - 65°) = 0.667 \div 0.466 = 1.43$, neglecting the effect of the cut-off. The cutoff will increase the factor of safety against sliding very materially. Walls of this type are always safe against overturning.

All bars must be embedded 50 diameters beyond the section of zero moment if ends are not hooked and 33 diameters if ends are hooked.

Working drawings based on the calculations given in the table are shown in the problem.

**Reference.**—For additional data on the design of retaining walls see the author's "The Design of Walls, Bins and Grain Elevators."

# CHAPTER XX.

## Design of Bridge Abutments and Piers.

**Introduction.**—An abutment is a structure that supports one end of a bridge span and at the same time supports the embankment that carries the track or roadway. An abutment also usually protects the embankment from the scour of the stream.

A pier is a structure that supports the ends of two bridge spans. Piers must be designed so as not to interfere with the flow of the stream, and care must be used to prevent undermining the pier by the scour of the stream.

**TYPES OF ABUTMENTS.**—Masonry abutments may be classified under four heads, Fig. 1, (*a*) straight or "stub" abutments; (*b*) wing abutments; (*c*) U abutments; (*d*) T abutments.

(*a*) The standard straight abutment of the N. Y. C. & H. R. R. R., shown in Fig. 1, is an excellent example of an abutment of this type. The earth fill is allowed to flow around the ends of the abutment as shown. Straight abutments should not be used where the water will wash the fill away.

(*b*) A standard wing abutment of the N. Y. C. & H. R. R. R. is shown in Fig. 1. The length of the wings is determined by the width of the roadway, the allowable slope of the sides of the embankment and the angle of the wings. The angle that the wings make with the face of the abutment ordinarily varies from 30 degrees to 45 degrees for standard conditions. For skew bridges and for unusual conditions the angle of the wing is variable.

(*c*) A standard U abutment of the N. Y. C. & H. R. R. R. is shown in Fig. 1. This is a wing abutment with the wings making an angle of 90 degrees with the face of the abutment. The wings are tied together by means of old railroad rails as shown. The wing walls run back into the fill, which flows down in front of the wings. If the slope is liable to be washed away by the scour of the stream the wings should be extended farther into the bank.

(*d*) A standard T abutment of the South Bend and Michigan Southern Railway for a skew span is shown in Fig. 1. The T abutment is essentially a straight abutment with a stem running back into the fill; the stem carries the roadway, supports the abutment, and prevents water from finding its way along the back of the abutment. A T abutment may be considered as a U abutment with the two wings in one.

**STABILITY OF BRIDGE ABUTMENTS WITHOUT WINGS.**—A bridge abutment must be stable (1) against overturning, (2) against sliding, and (3) against crushing the material on which the abutment rests, or the masonry in the abutment. The problem of the design of a bridge abutment is essentially the same as the design of a retaining wall, for which see Chapter V. The method of design will be shown by giving the calculations for a straight concrete abutment for West Alameda Avenue Subway, Denver, Colo.

**Design of Concrete Abutment for West Alameda Avenue Subway, Denver, Colorado.**—The height of the abutment is 21 ft. 6 in. from the bottom of the footing to the top of the bridge seat, and 25 ft. 0¾ in. to the top of the back wall. The following assumptions were made: Weight of concrete, 150 lb. per cu. ft.; weight of filling, $w = 100$ lb. per cu. ft.; angle of repose of the filling, 1½ to 1 ($\phi = 33° 42'$); surcharge 800 lb. per sq. ft., equivalent to 8 ft. of filling; maximum load on foundation 6,000 lb. per sq. ft.

**Solution.**—After several trials the dimensions given in Fig. 2 were taken. The stability of the abutment was investigated for two conditions: (*a*) with a full live and dead load on the bridge and on the filling, and (*b*) with no live load on the bridge and no surcharge coming on the filling above the wall, it being assumed that a locomotive is approaching the bridge from the right, and

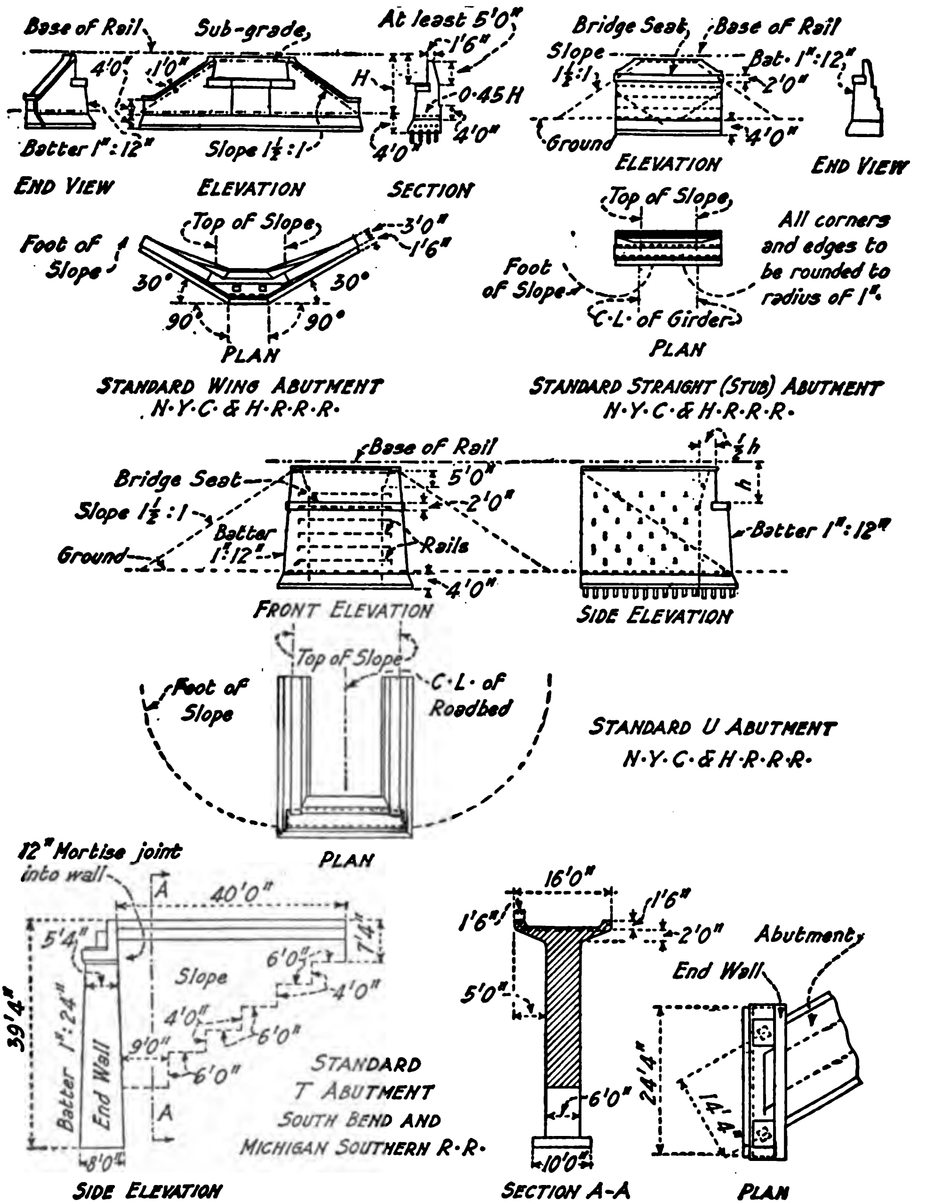



FIG. 1. TYPES OF MASONRY ABUTMENTS.

has reached the point 2 in (*b*), Fig. 2. The weight of the girders and the live load was assumed as uniformly distributed over a length of the abutment equal to the distance between track centers, and one lineal foot of wall was investigated.

**Case (a).**—The pressure of the filling on the plane $B$–2 was calculated as in Chapter V, Fig. 9, and is $P' = 14{,}700$ lb., acting through the center of gravity of the trapezoid 2–3–4–$B$. The weight of the filling and surcharge is $W_2 + W_3 = 14{,}900$ lb., which when combined with $P'$ gives the resultant pressure of the filling on the wall $= P = 20{,}900$ lb. The pressure $P$ is then combined with the weight of the wall, $W_1 = 29{,}800$ lb., and with the dead load and live load from the girder $= 12{,}820$ lb., giving the resultant pressure on the foundation, $E = 59{,}400$ lb., and acting, $b = 1.4$ ft. from the center of the wall, and $F = 57{,}500$ lb.

1. *Stability Against Overturning.*—The resultant $E$ is nearly vertical and well within the middle third, so that the wall is amply safe against overturning.

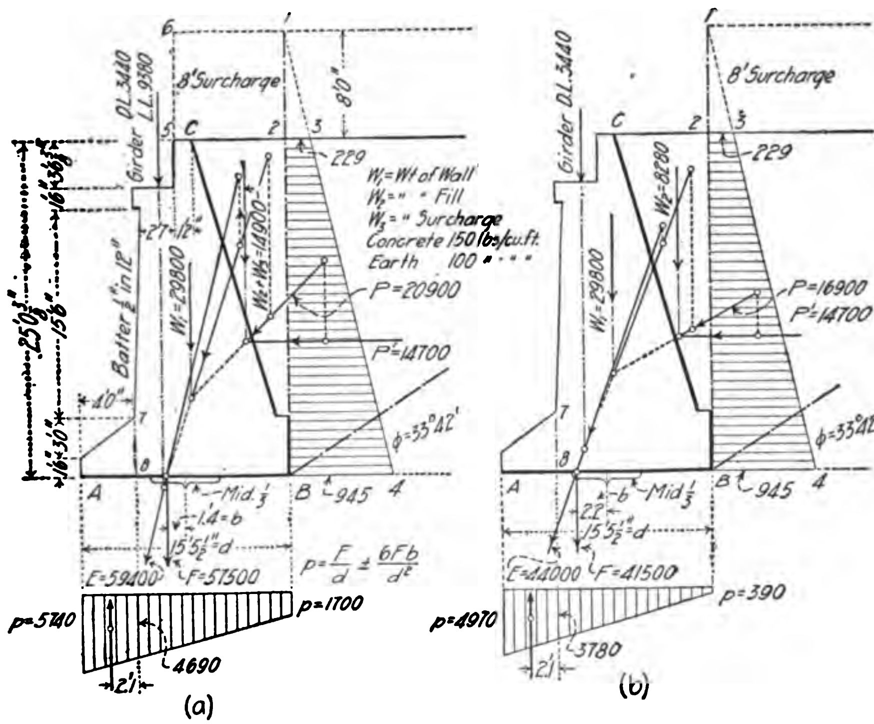



FIG. 2. ABUTMENT FOR WEST ALAMEDA AVENUE SUBWAY, DENVER, COLO.

2. *Stability Against Sliding.*—Assuming that $\phi' = 30°$, then the coefficient of friction will be $\tan \phi' = 0.57$. Using the definition of factor of safety given in equation (27) Chapter V. the resistance of the wall against sliding will be $57{,}500 \times 0.57 = 32{,}765$ lb. The sliding force is $P' = 14{,}700$ lb., and the factor of safety is $32{,}765/14{,}700 = 2.23$, which is ample.

3. *Pressure on Foundation.*—The pressure on the foundation will be $p = F/d \pm 6F \cdot b/d^2 = +\ 5{,}740$ and $+\ 1{,}700$ lb. per sq. ft., which is safe.

4. *Upward Pressure on Front Projection of Foundation.*—The base will be investigated on the plane 7–8 to see that the upward pressure will not break off the front projection of the foundation. The bending moment of the upward pressure about the front face of the wall in (*a*), Fig. 2, will be

$$M = \tfrac{1}{2}(5{,}740 + 4{,}690)4 \times 2.1 \times 12$$
$$= 525{,}672 \text{ in-lb.}$$

The tension on the concrete at the bottom of the footing will be

$$f = \frac{M \cdot c}{I} = \frac{M \cdot d}{2I} = \frac{525{,}672 \times 27}{157{,}464}$$
$$= 92 \text{ lb. per sq. in.}$$

The footing is safe, but $\frac{3}{4}$ in. □ rods were placed 18 in. centers and 3 in. from the bottom of the foundation.

**Case (b).**—The solution is the same as for (*a*) except that the live load from the girder = 9,980 lb., and the surcharge load 1–2–5–6 = $W_3$ = 6,620 lb. were omitted. The wall is safe for overturning. The factor of safety against sliding is from equation (27) Chapter XIX, $f_s = 41{,}500 \times 0.57/14{,}700 = 1.6$, which is safe. The pressure on the foundation is safe.

The back wall was placed after the bridge seats were finished. To bond the back wall to the abutment, $\frac{1}{2}$ in. □ rods 4 ft. long, spaced 18 in. centers, were placed in two rows 3 in. from the back and front face, one-half of the length of the rod being imbedded in the main wall.

**PRINCIPLES OF DESIGN.**—To prevent tension on the back side of the footing and to make sure that the maximum compression on the front side of the footing shall not be greater than twice the average pressure, the resultant of the thrust of the filling, the weight of the masonry, the weight of the bridge and the live load must strike within the middle third of the base. Where the abutment rests on rock or solid material where settlement will not occur, it will not be serious if the resultant strikes a little outside of the middle third, providing the allowable pressure on the foundation is not exceeded. When the abutment is on compressible material where settlement will take place, the resultant of the pressures should strike at or back of the center of the base, so that the abutment will not tip forward in settling. It is standard practice to use piles in the foundation for abutments resting on compressible soil.

For the design of wing walls see the design of Retaining Walls, Chapter XIX.

In addition to the requirements for stability abutments should satisfy the following additional requirements.

(*a*) The abutment should protect the bank from scour. (*b*) The abutment should prevent the embankment drainage from washing away the bank. (*c*) The abutment should be easily drained.

**Empirical Design.**—A common rule is to make the minimum thickness of the main part of the abutment not less than $\frac{4}{10}$ the height above any section; and project the footings on each side as may be required. Empirical methods of design often give unsatisfactory results and are not to be recommended.

**DESIGN OF BRIDGE PIERS.**—Bridge piers must be designed (1) for the total vertical load due to the dead load of the span and the live load on the span, and the weight of the pier; (2) for wind pressure on the pier and the bridge; (3) to withstand floating drift and ice; and (4) to take the longitudinal thrust due to stopping a car or train on the bridge, and due to temperature when the rollers do not move freely. The wind pressures are calculated as specified in specifications for bridges, and are assumed to act in the vertical line of the center of the pier; on the top chord of the truss; the bottom chord of the truss; 6 or 7 feet above the base of the rail; and at the center of gravity of the exposed part of the pier. The total wind moment is then calculated about the leeward edge of the base of the pier, and the maximum stresses on the foundation due to direct load and wind are calculated in the same manner as the calculation of the pressures of abutments.

The effect of the current of the stream and of floating ice and drift are difficult to calculate. The pressure of a flowing stream on an obstruction is given by the formula

$$P = m \cdot w \cdot a \cdot \frac{V^2}{2g}$$

where $P$ = the total pressure on the surface; $m$ = a constant; $w$ = weight of a cubic foot of water; $a$ = area of wetted surface normal to the current in square feet; $v$ = velocity of current in feet per second; and $g$ = acceleration due to gravity = 32.2 feet. The value of $m$ varies with the shape and the dimensions of the pier. Weisbach's Mechanics gives the following data:— For a prism three times as long as broad, $m$ = 1.33. For a pier five or six times as long as broad and with a cutwater having plane faces and an angle of 30 degrees between the cutwater faces, $m$ = 0.48. For a square pier, $m$ = 1.28, and for a circular pier, $m$ = 0.64.

The maximum pressure due to floating ice will be the crushing strength of the ice, which varies from 400 to 800 lb. per sq. in. The principal danger from floating ice and drift is that the current of the stream will be deflected downward and will gouge out the material around and under the pier and cause failure. To prevent this it is quite common to build piers with a "break-water," "starkwater," "cutwater," or nose that will deflect drift and ice, or to put in a pile protection on the upstream side of the pier. If the water can get under the pier the buoyancy of the water must be considered in calculating the stability of the pier. If there is danger of scouring then it is well to deposit large stones and riprap around the base of the pier.

**Batter.**—Piers and abutments are seldom battered more than one inch to one foot of vertical height, or less than one-half inch to the foot, although high piers are sometimes battered only one-fourth inch to one foot.

**ALLOWABLE PRESSURES ON FOUNDATIONS.**—The allowable pressures on foundations depend upon the material, the drainage, the amount of lateral support given by the adjacent material, the depth of the foundation, and other conditions, so that it is not possible to give data that will be more than an aid to the judgment. If properly designed a moderate settlement of some particular structure may do no harm, while a less settlement in another structure may be disastrous. Professor I. O. Baker gives the values in Table I in his " Masonry Construction."

TABLE I.

SAFE BEARING POWER OF SOILS.*

| Kind of Material. | Safe Bearing Power in Tons per Square Foot. Min. | Max. |
|---|---|---|
| Rock hardest in thick layers in bed | 200 | — |
| Rock equal to best ashlar masonry | 25 | 30 |
| Rock equal to best brick | 15 | 20 |
| Rock equal to poor brick | 5 | 10 |
| Clay in thick beds, always dry | 4 | 6 |
| Clay in thick beds, moderately dry | 2 | 4 |
| Clay soft | 1 | 2 |
| Gravel and coarse sand, well cemented | 8 | 10 |
| Sand compact and well cemented | 4 | 6 |
| Sand clean, dry | 2 | 4 |
| Quicksand, alluvial soils, etc. | 0.5 | 1 |

Present practice is more nearly given by the values in Table II. Foundations should never be placed directly on quicksand.

TABLE II.

ALLOWABLE BEARING ON FOUNDATIONS.

| Kind of Material. | Tons per Square Foot. |
|---|---|
| Soft clay or loam | 1 |
| Ordinary clay and dry sand mixed with clay | 2 |
| Dry sand and dry clay | 3 |
| Hard clay and firm, coarse sand | 4 |
| Firm, coarse sand and gravel | 6 |
| Shale rock | 8 |
| Hard rock | 20 |

* Baker's " Masonry Construction," John Wiley & Sons.

Mr. E. L. Corthell gives the summary of the pressures on deep foundations in Table III.

TABLE III.

ACTUAL PRESSURES ON DEEP FOUNDATIONS.*

| Material. | Number of Examples. | Pressure in Tons per Square Foot. | | |
|---|---|---|---|---|
| | | Maximum. | Minimum. | Average. |
| Actual Pressures which Showed No Settlement. | | | | |
| Fine sand................. | 10 | 5.4 | 2.25 | 4.5 |
| Coarse sand and gravel..... | 33 | 7.75 | 2.4 | 5.1 |
| Sand and clay............. | 10 | 8.5 | 2.5 | 4.9 |
| Alluvium and silt.......... | 7 | 6.2 | 1.5 | 2.9 |
| Hard clay................. | 16 | 8.0 | 2.0 | 5.08 |
| Hard pan................. | 5 | 12.0 | 3.0 | 8.7 |
| Actual Pressures which Showed Settlement. | | | | |
| Fine sand................. | 3 | 7.0 | 1.8 | 5.2 |
| Clay...................... | 5 | 5.6 | 4.5 | 5.2 |
| Alluvium and silt.......... | 2 | 7.6 | 1.6 | ... |
| Sand and clay............. | 3 | 7.4 | 1.6 | 3.3 |

The data in Table III shows that great care must be used in determining on the allowable pressure for any particular foundation, and that safe values for the bearing power of soils should only be used as an aid to the judgment of the engineer.

**WATERWAY FOR BRIDGES.**—The clear waterway for bridges should be ample; great care should be used to prevent floating logs and debris from clogging up the opening. The necessary waterway depends upon the character and size of the runoff area, the slope and size of the stream and upon other local conditions.

Many formulas have been proposed for determining the waterway of culverts and bridges. The formula best known to the author is that proposed by Professor A. N. Talbot. It is

$$A = c\sqrt[4]{M^3}$$

where $A$ = area of the required opening in sq. ft.;
$M$ = area of drainage basin in acres;
$c$ = a coefficient varying with the slope of the ground, slope of the drainage area, character of the soil and character of vegetation.

Professor Talbot gives the following values of $c$ : $c = \frac{2}{3}$ to 1 for steep and rocky ground; $c = \frac{1}{3}$ for rolling agricultural country, subject to floods at times of melting snow, and with the length of valley 3 to 4 times its width; $c = \frac{1}{5}$ to $\frac{1}{6}$ for districts not affected by accumulated snow and where the length of the valley is several times its width.

The "Dun Drainage Table," which is quite generally used by railways, is given on page 251, of the author's "Structural Engineers' Handbook."

**PREPARING THE FOUNDATIONS.**—The preparation of the site of the abutment or pier will depend upon the conditions and character of the material.

**Rock.**—Where the water can be excluded, the rock should be cleared of all overlying material and disintegrated rock. The surface is then leveled up either by cutting off the projections or by depositing a layer of concrete.

* "Allowable Pressures on Deep Foundations" by E. L. Corthell, John Wiley & Sons.

**Hard Ground**.—The material should be excavated well below the frost and scour line. Where the foundations cannot be carried low enough to prevent undermining, piles should be driven at about 2½ to 3 ft. centers over the foundation.

## TABLE IV.

### Waterway for Bridges and Culverts.

Square Feet of Waterway Required for Culverts and Bridges with Varying Size and Character of Tributary Watershed, Calculated by Talbot's formula*:

| Area, Acres. | Area of Culvert, Sq. Ft. | | | Area, Acres. | Area of Culvert, Sq. Ft. | | | Area, Sq. Mi. | Area of Culvert, Sq. Ft. | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | $c = 1$. | $c = 1/3$. | $c = 1/5$. | | $c = 1$. | $c = 1/3$. | $c = 1/5$. | | $c = 1$. | $c = 1/3$. | $c = 1/5$. |
| 1 | 1.0 | 0.3 | 0.2 | 30 | 12.8 | 4.3 | 2.6 | ½ | 75 | 25 | 15 |
| 2 | 1.7 | 0.6 | 0.3 | 40 | 15.9 | 5.3 | 3.2 | ¾ | 103 | 34 | 21 |
| 3 | 2.3 | 0.8 | 0.5 | 60 | 21.6 | 7.2 | 4.3 | 1 | 127 | 42 | 25 |
| 4 | 2.8 | 0.9 | 0.6 | 80 | 26.8 | 8.9 | 5.4 | 2 | 214 | 71 | 43 |
| 5 | 3.3 | 1.1 | 0.7 | 100 | 31.6 | 10.5 | 6.3 | 3 | 290 | 97 | 58 |
| 10 | 5.6 | 1.9 | 1.1 | 120 | 36.0 | 12.0 | 7.0 | 4 | 356 | 119 | 71 |
| 15 | 7.6 | 2.5 | 1.5 | 160 | 45.0 | 15.0 | 9.0 | 6 | 488 | 163 | 98 |
| 20 | 9.5 | 3.2 | 1.9 | 240 | 61.0 | 20.0 | 12.0 | 10 | 715 | 238 | 143 |

* $A = c\sqrt[4]{M^3}$ where $A$ = sectional area of culvert in square feet.
$M$ = area of tributary watershed in acres.
$c$ = 1 for abrupt slopes.
1/3 for rolling agricultural country.
1/5 for level cultivated land.

**Soft Ground.**—The materials should be excavated to a solid stratum or piles spaced about 2½ to 3 ft. centers should be driven over the foundation to a good refusal. The piles should be cut off below low water level to carry a timber grillage, or concrete may be deposited around the heads of the piles. Where water cannot be excluded it will be necessary to use one of the following methods: open caisson, crib, coffer dam, or pneumatic caisson.

In using an open caisson the masonry is built up or the concrete is deposited in a water tight box built of heavy timbers or of reinforced concrete, the caisson being sunk as the pier is built up. The caisson is commonly floated into place and then is sunk on piles which have been sawed off to receive it, or on a solid rock foundation. The sides of timber caissons are usually removed after the pier is completed.

Timber cribs are made of squared timbers placed transversely and longitudinally, and bolted together so as to form a solid structure with open pockets. The crib is sunk by loading the pockets with stone. No timber should be left above the low water mark in open caissons or cribs.

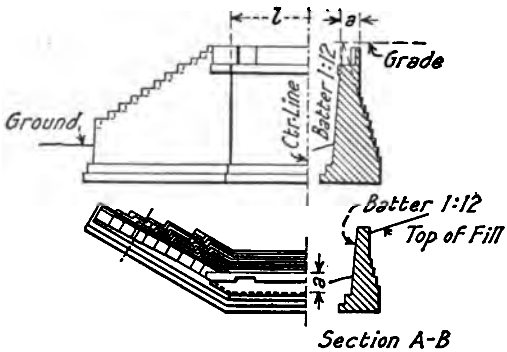
A coffer dam is usually made by driving two rows of sheet piling around the pier, the space between the rows of piling being filled with clay puddle. For small depths a single row of sheet piling is often sufficient. Where the depth is too great for one length of sheet piling, additional rows are driven inside the first. Steel sheet piling is now much used for difficult foundations. Steel sheet piling can be driven through ordinary drift and similar material, is not limited in depth, and is practically water tight when used in a single row. It can be drawn and used again. It is almost impossible to shut off all the water with a coffer dam, and pumps should be provided.

Pneumatic caissons should only be used under the direction of experienced engineers and will not be considered here.

For details of sinking piers see Jacoby & Davis' "Foundations of Bridges and Buildings", McGraw-Hill Book Company.

**EXAMPLES OF BRIDGE ABUTMENTS AND PIERS.**—Several different types of bridge abutments and piers will be described in detail.

**Cooper's Standard Abutments.**—The abutment in (a), Fig. 3, is from Cooper's "General Specifications for Foundations and Substructures of Highway and Electric Railway Bridges." The length $l$, and the thickness, $a$, for highway and single track electric railway bridges are as given, and are proportional for intermediate spans. These abutments may be made of either first-class stone masonry or first-class Portland cement concrete.

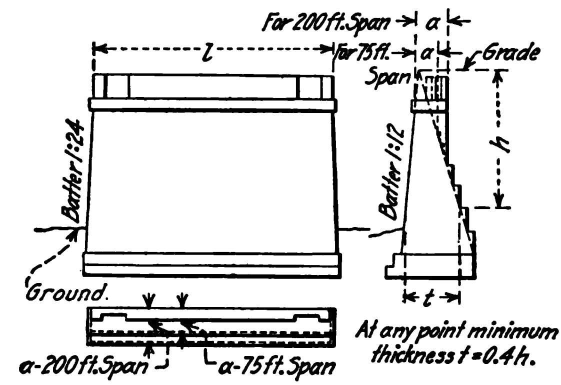



(a) HIGHWAY ABUTMENT WITH WING WALLS

DIMENSIONS OF MASONRY ABUTMENTS WITH WING WALLS

| Distance, a | Span, Feet | Length, l. |
|---|---|---|
| 2'6" | 50 | w+4'0" |
| 2'8" | 100 | w+5'0" |
| 3'0" | 150 | w+5'9" |
| 3'4" | 200 | w+6'6" |
| 3'6" | 250 | w+7'0" |

w = center to center of trusses, 14 ft. for single track, 26 ft. for double track.




(b) HIGHWAY ABUTMENT WITHOUT WING WALLS

APPROXIMATE QUANTITIES IN CU. YDS. OF ONE MASONRY ABUTMENT WITHOUT WING WALLS

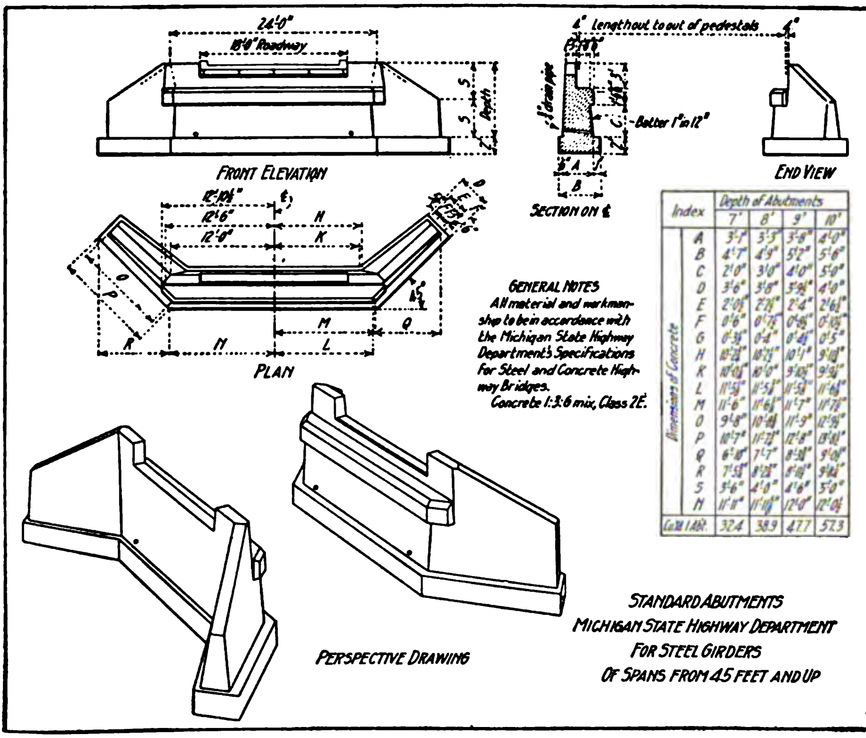
| Span Feet | Roadway | Depth Footing Below Grade | | | | |
|---|---|---|---|---|---|---|
| | | 10' | 15' | 20' | 25' | 30' |
| 100 | 12 Feet | 20 | 39 | 67 | 100 | 145 |
| | 20 Feet | 28 | 56 | 95 | 145 | 206 |
| | E, Single T. | 21 | 44 | 75 | 112 | 160 |
| | E, Double T. | 36 | 72 | 120 | 183 | 260 |
| 300 | 12 Feet | 22 | 45 | 77 | 116 | 165 |
| | 20 Feet | 31 | 63 | 106 | 161 | 227 |
| | E, Single T. | 25 | 50 | 85 | 128 | 181 |
| | E, Double T. | 49 | 84 | 141 | 210 | 296 |

FIG. 3. MASONRY ABUTMENTS FOR ELECTRIC RAILWAY AND HIGHWAY BRIDGES. COOPER'S STANDARDS.

For double track electric railway bridges add one foot to the value of $a$ in Fig. 3. The minimum thickness of the wall at any point is to be 0.4 of the height. The length of the wing walls will be determined by local conditions.

The abutment without wing walls in (b), Fig. 3, has the same dimensions as the abutment with wing walls. The width for single track electric railways may be taken as 14 ft., double track 26 ft. The approximate cubical quantities in abutments without wing walls are given in Fig. 3.

**Michigan Highway Commission.**—Standard plans for plain concrete abutments as prepared by the Michigan Highway Commission are given in Fig. 4. The concrete is to be 1–3–6 mix with a gravel, hard limestone or hard sandstone aggregate. Foundations are to be well below




| Index | Depth of Abutments 7' | 8' | 9' | 10' |
|---|---|---|---|---|
| A | 3'-1" | 3'-3" | 3'-8" | 4'-0" |
| B | 4'-7" | 4'-9" | 5'-2" | 5'-6" |
| C | 2'-0" | 3'-0" | 4'-0" | 5'-0" |
| D | 3'-6" | 3'-8" | 3'-9½" | 4'-0" |
| E | 2'-0½" | 2'-2½" | 2'-4" | 2'-6½" |
| F | 0'-6" | 0'-7½" | 0'-9½" | 0'-10½" |
| G | 0'-3½" | 0'-4" | 0'-4½" | 0'-5" |
| H | 10'-2½" | 10'-2½" | 10'-1" | 9'-10½" |
| K | 10'-0½" | 10'-0" | 9'-10½" | 9'-9½" |
| L | 11'-5½" | 11'-5¾" | 11'-5½" | 11'-6½" |
| M | 11'-6" | 11'-6½" | 11'-7" | 11'-7½" |
| O | 9'-8" | 10'-8½" | 11'-9" | 12'-9½" |
| P | 10'-7" | 11'-7½" | 12'-8" | 13'-8½" |
| Q | 6'-8" | 7'-7" | 8'-3½" | 9'-0½" |
| R | 7'-5½" | 8'-2½" | 8'-10" | 9'-6½" |
| S | 3'-6" | 4'-0" | 4'-6" | 5'-0" |
| N | 11'-11" | 11'-11½" | 12'-0" | 12'-0½" |
| Cu. Yd. 1 Abt. | 32.4 | 38.9 | 47.7 | 57.3 |

FIG. 4. STANDARD MASONRY ABUTMENTS FOR STEEL HIGHWAY SPANS. MICHIGAN HIGHWAY COMMISSION.

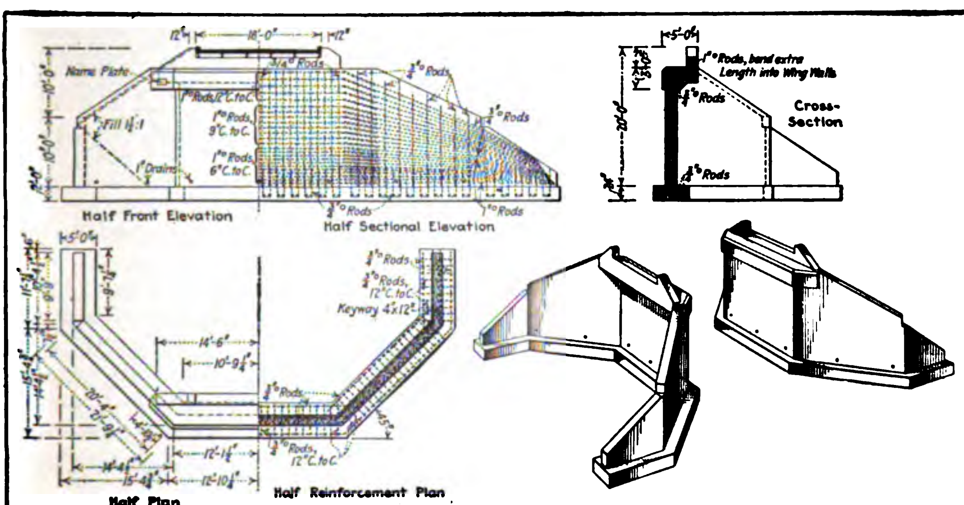



FIG. 5. REINFORCED CONCRETE HIGHWAY BRIDGE ABUTMENTS. MICHIGAN HIGHWAY COMMISSION.

scour with a minimum of 3 feet, if above water. Where piles are necessary, they are to be driven not less than 10 ft. deep.

Plans for reinforced concrete abutments designed by the Michigan Highway Commission are given in Fig. 5. These abutments are designed for an 18-ft. roadway, and a height of 20 feet. The concrete is to be 1–2–4 mix. The two abutments contain 100 cu. yd. concrete and 16,000 lb. reinforcing steel. The wings and the body of the abutment are designed to act together as a

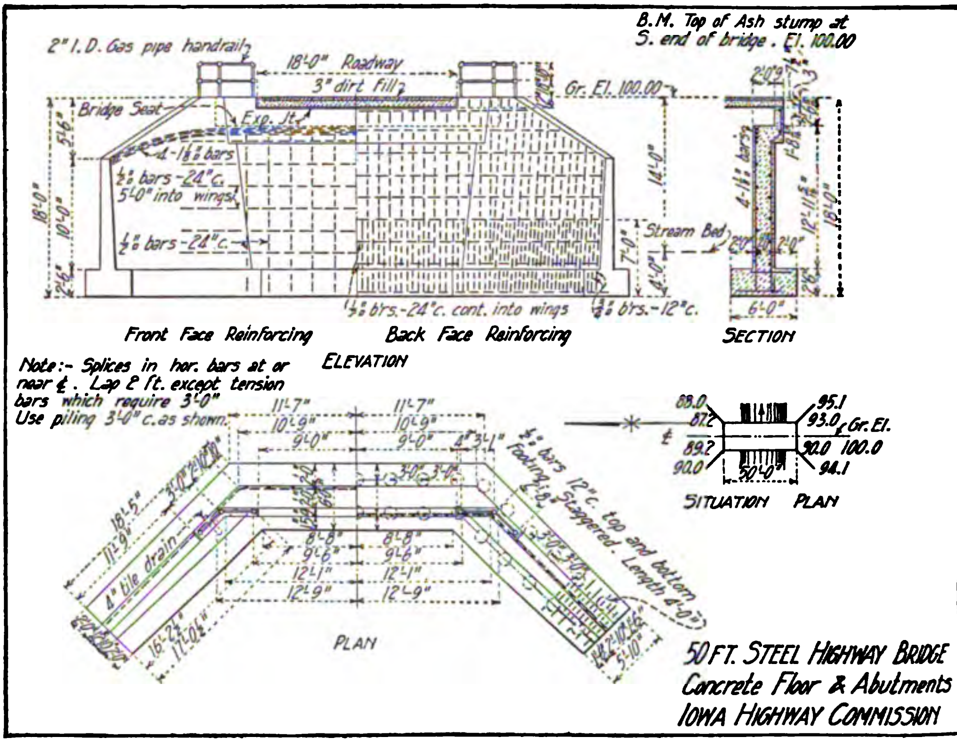



FIG. 6. REINFORCED CONCRETE HIGHWAY BRIDGE ABUTMENTS. IOWA HIGHWAY COMMISSION.

unit. The fill is drained through the abutment by means of seven one-inch drains in each abutment. The Commission has prepared plans for abutments for 16-ft. and 18-ft. roadways, for heights of 10 ft. to 20 ft., varying by one foot.

**Iowa Highway Commission.**—Reinforced concrete abutments for a 50-ft. span steel highway bridge are given in Fig. 6; and reinforced concrete abutments for an 80-ft. span steel bridge are given in Fig. 7. These abutments were designed by the Iowa Highway Commission. The wings make an angle of 45 degrees with the face of the abutment, and are tied to the body of the abutment by means of reinforcement. The width of base is taken equal to one-third the total height of the abutment. Piles are to be driven, where needed, in the manner indicated in the drawings.

**Illinois Highway Commission.**—Details of reinforced concrete abutments for a concrete girder span are shown in Fig. 8. The abutments are of the slab type and are designed to act as a unit. The wings are designed to act as cantilever walls, and are also tied to the body of the abutment. The concrete girders are supported at one end on expansion rockers placed in a recess in the top

of the abutment as shown on the drawings. The wings are shown as making an angle of 45 degrees with the face of the abutment.

Details of a U-abutment are shown in Fig. 9. The wings are tied together with steel bars which are protected by imbedding in concrete. A view of a concrete slab bridge with a U-abutment is shown in Fig. 1, Chapter XVII.

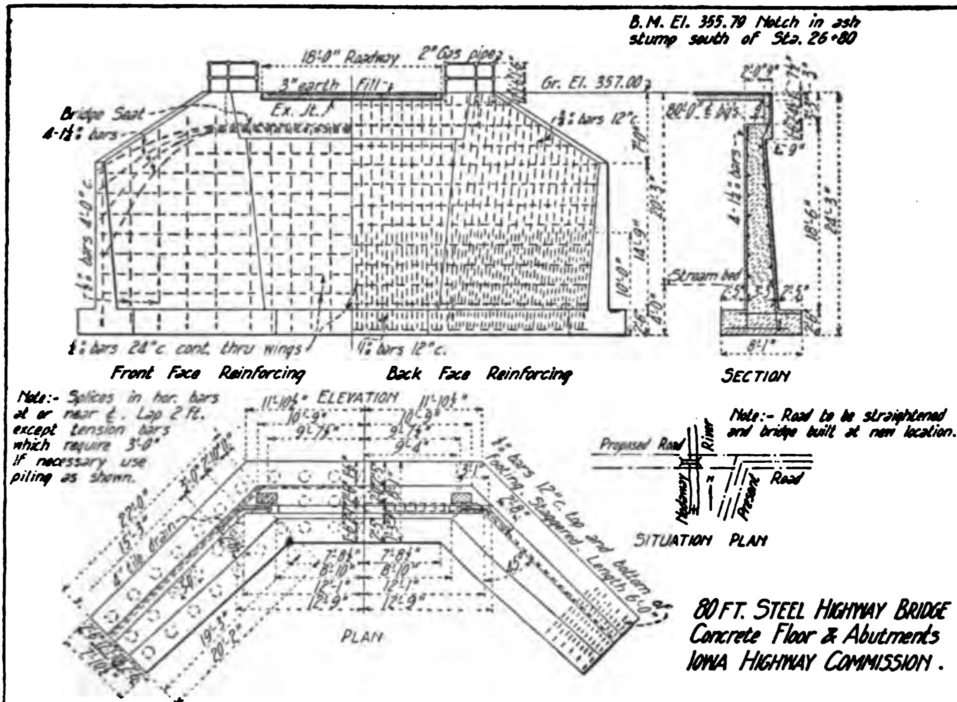


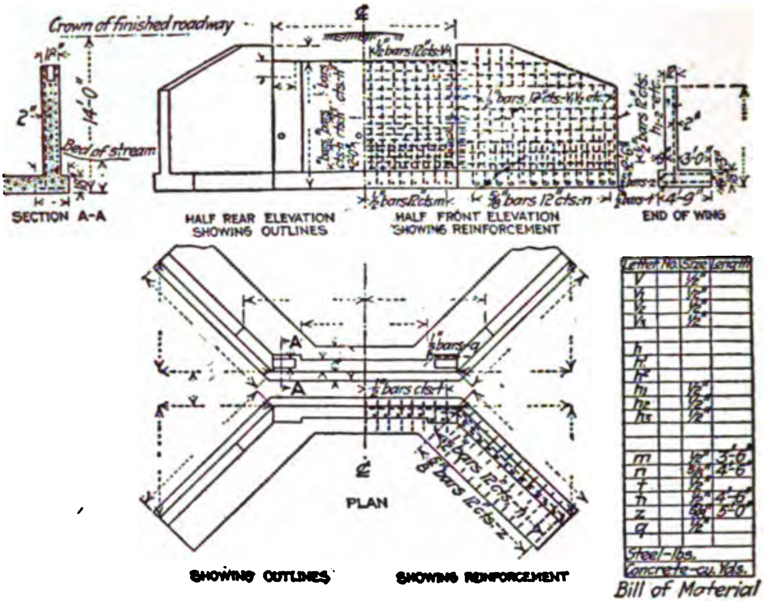

FIG. 7. REINFORCED CONCRETE HIGHWAY BRIDGE ABUTMENTS. IOWA HIGHWAY COMMISSION.

**Pedestal Abutment.**—Details of a pedestal abutment used for a 150-ft. span steel highway bridge over the New York Barge Canal are shown in Fig. 10. The abutment consists of a forward pier with two pedestals resting on the body of the pier, and having a connecting wall to prevent the fill from sliding into the canal, and two rear columns.

The tops of the pier and the columns are connected by means of transverse beams which carry the concrete floor slab. This abutment is very economical and in addition gives an increased waterway for flood flow.

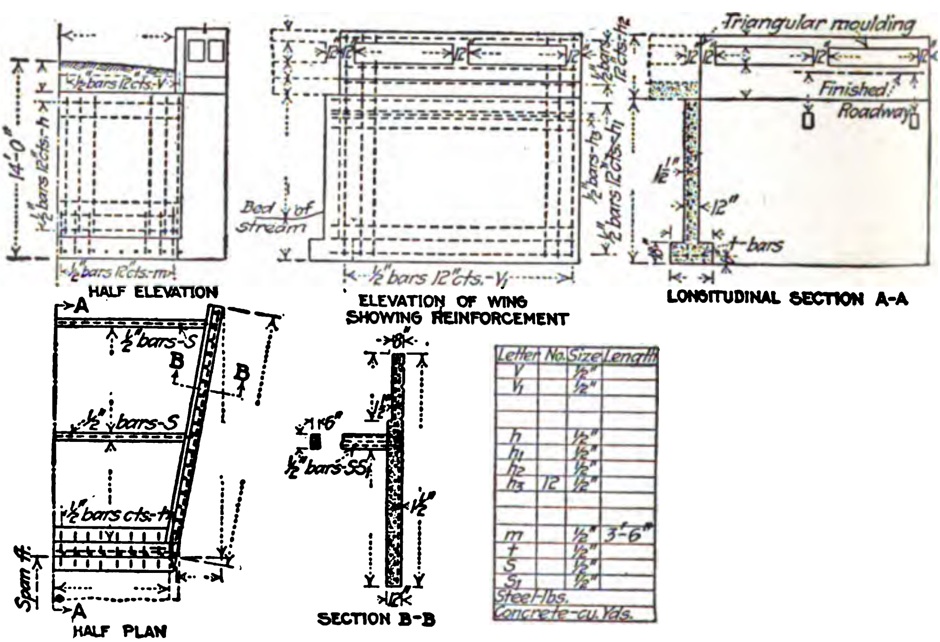
**Cooper's Standard Masonry Piers.**—The masonry pier in Fig. 11 is from Cooper's "General Specifications for Substructures of Highway and Electric Railway Bridges." The length, *l*, and the thickness, *a*, for highway and single track electric railway bridges are given in Fig. 11. These piers may be made of either first-class stone masonry, or first-class Portland cement concrete.

For double track electric railway bridges add one foot to *l*, and 6 inches to *a*. The width $w$ = center to center of trusses, and may ordinarily be taken 14 ft. for single track, and 26 ft. for double track through bridges. Where drift and logs are liable to injure the pier the nose of the cut-water should be protected with a steel angle or plate. The approximate cubical contents of the piers are given in Fig. 11.




| Letter | No. | Size | Length |
|---|---|---|---|
| V | | 1/2" | |
| $V_1$ | | 1/2" | |
| $V_2$ | | 1/2" | |
| $V_3$ | | 1/2" | |
| | | | |
| h | | | |
| h' | | | |
| h" | | | |
| $h_1$ | | 1/2" | |
| $h_2$ | | 1/2" | |
| $h_3$ | | 1/2" | |
| | | | |
| | | | |
| m | | 1/2" | 3'-6" |
| n | | 5/8" | 4'-6" |
| t | | 1/2" | |
| h | | 1/2" | 4'-6" |
| z | | 5/8" | 5'-0" |
| q | | 1/2" | |
| | | | |
| Steel-lbs. | | | |
| Concrete-cu. Yds. | | | |

FIG. 8. STANDARD ABUTMENTS FOR A CONCRETE GIRDER BRIDGE. ILLINOIS HIGHWAY COMMISSION.




| Letter | No. | Size | Length |
|---|---|---|---|
| V | | 1/2" | |
| $V_1$ | | 1/2" | |
| | | | |
| | | | |
| h | | 1/2" | |
| $h_1$ | | 1/2" | |
| $h_2$ | | 1/2" | |
| $h_3$ | 12 | 1/2" | |
| | | | |
| | | | |
| m | | 1/2" | 3'-6" |
| t | | 1/2" | |
| S | | 1/2" | |
| $S_1$ | | 1/2" | |
| Steel-lbs. | | | |
| Concrete-cu. Yds. | | | |

FIG. 9. U-ABUTMENT. ILLINOIS HIGHWAY COMMISSION.

**Michigan State Highway Commission. Masonry Piers.**—Details of standard masonry piers as designed by Michigan State Highway Commission are given in Fig. 12. Quantities of concrete as shown in the cut.

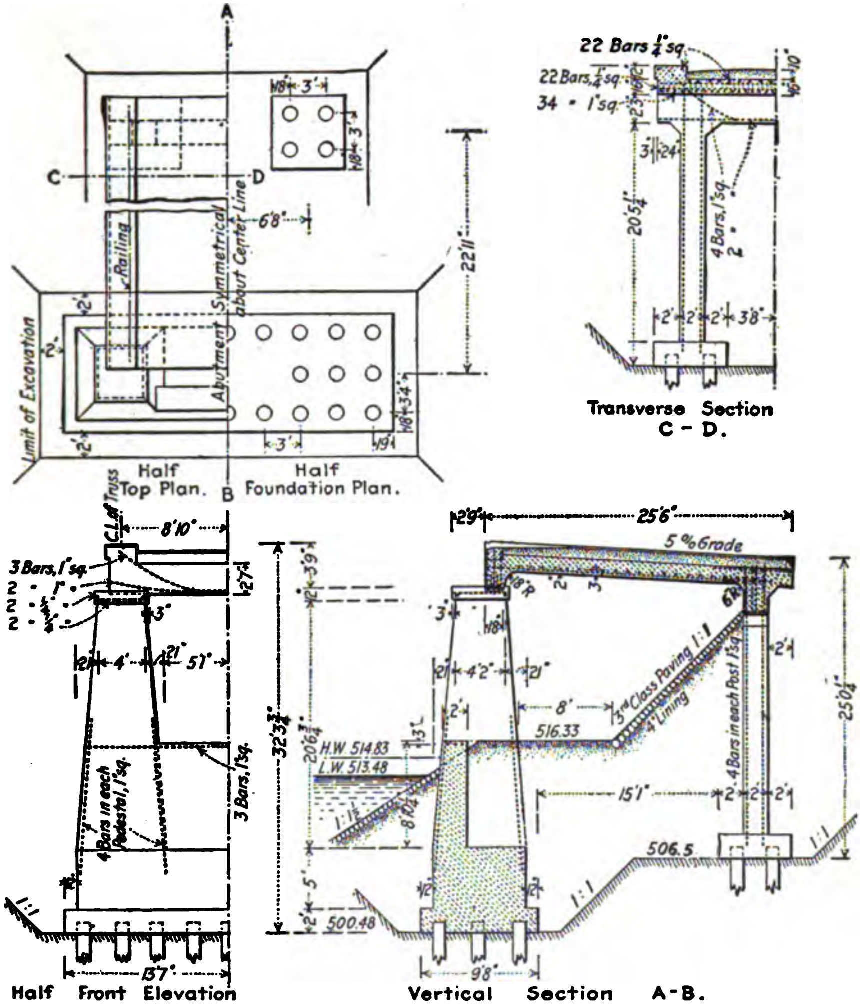
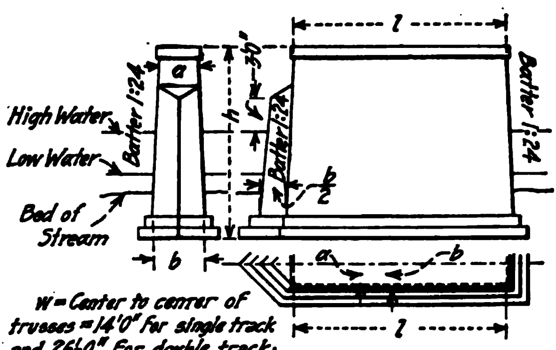



FIG. 10. DETAILS OF A PEDESTAL ABUTMENT ON BARGE CANAL.
(Engineering News, August 18, 1910.)

**Iowa Highway Commission. Masonry Piers.**—Details of standard masonry piers designed by the Iowa Highway Commission are shown in Fig. 13. The upstream end of the pier is provided with a cut-water protected by an 8 in. by 8 in. angle. The bridge seat of the pier is reinforced with steel bars. The widths and lengths of pier for different conditions are given in the cut. A typical piling plan is also shown.

**Pedestal Piers.**—A reinforced concrete pedestal pier designed by the Minnesota Highway Commission is shown in Fig. 14. This pier was designed to carry a steel beam highway bridge. Details of the pier are shown in the cut.




HIGHWAY PIERS

DIMENSIONS FOR MASONRY PIER FOR HIGHWAY AND SINGLE TRACK ELECTRIC RAILWAY BRIDGES

| Distance a | Span Feet | Length l |
|---|---|---|
| 2'8" | 50 | w+4'0" |
| 2'10" | 75 | w+4'6" |
| 3'2" | 100 | w+5'0" |
| 3'8" | 150 | w+5'9" |
| 4'4" | 200 | w+6'6" |
| 4'10" | 250 | w+7'0" |
| 5'4" | 300 | w+7'6" |

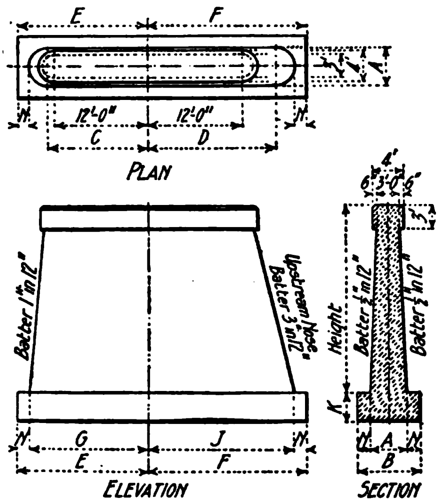
For double track Electric Railway bridges add 12" to l and 6" to a.

APPROXIMATE CONTENTS IN CUBIC YARDS OF ONE MASONRY PIER

| Spans Feet | Roadway | Depth of Pier from Top of Coping to Bottom of Footing in Feet. 10 | 15 | 20 | 25 | 30 |
|---|---|---|---|---|---|---|
| 100 | 12 Feet | 29 | 44 | 60 | 77 | 94 |
| | 20 Feet | 38 | 59 | 82 | 108 | 136 |
| | E, Single T. | 31 | 46 | 62 | 80 | 100 |
| | E, Double T. | 50 | 75 | 102 | 132 | 166 |
| 150 | 12 Feet | 34 | 51 | 70 | 90 | 111 |
| | 20 Feet | 46 | 70 | 95 | 125 | 157 |
| | E, Single T. | 37 | 54 | 74 | 96 | 120 |
| | E, Double T. | 58 | 86 | 118 | 153 | 191 |
| 200 | 12 Feet | 39 | 58 | 80 | 103 | 128 |
| | 20 Feet | 53 | 80 | 109 | 143 | 178 |
| | E, Single T. | 43 | 63 | 86 | 112 | 140 |
| | E, Double T. | 66 | 99 | 135 | 174 | 217 |
| 250 | 12 Feet | 44 | 66 | 90 | 116 | 145 |
| | 20 Feet | 61 | 91 | 123 | 160 | 199 |
| | E, Single T. | 48 | 74 | 98 | 127 | 159 |
| | E, Double T. | 73 | 109 | 149 | 192 | 238 |
| 300 | 12 Feet | 49 | 73 | 100 | 130 | 162 |
| | 20 Feet | 68 | 101 | 137 | 177 | 220 |
| | E, Single T. | 54 | 80 | 109 | 142 | 178 |
| | E, Double T. | 80 | 120 | 164 | 210 | 260 |

FIG. 11. MASONRY PIERS FOR HIGHWAY AND ELECTRIC RAILWAY BRIDGES. COOPER'S STANDARDS.

Details of a concrete pedestal pier designed by the Illinois Highway Commission are shown in Fig. 15. This pier was designed to support one end of the 224-ft. steel span and one end of a 45-ft. span reinforced concrete through girder in the Vermillion Bridge. A view of this bridge showing the piers is given in Fig. 4, Chapter XVII. Details of the expansion joint in the floor between the two spans are shown in the cut.




CONCRETE IN PIER - CU. YD.

| Height in Ft. | 1 Ft. Section | Short Side | Long Side | Total |
|---|---|---|---|---|
| 6 | 1.21 | 16.8 | 17.1 | 33.9 |
| 8 | 1.51 | 21.1 | 21.8 | 42.9 |
| 10 | 1.84 | 25.7 | 26.9 | 52.6 |
| 12 | 2.18 | 30.7 | 32.5 | 63.2 |
| 14 | 2.54 | 35.9 | 38.6 | 74.5 |
| 16 | 2.91 | 41.5 | 45.2 | 86.7 |
| 18 | 3.30 | 47.4 | 52.3 | 99.7 |
| 20 | 3.71 | 53.6 | 59.8 | 113.4 |
| 22 | 4.13 | 60.2 | 68.0 | 128.2 |
| 24 | 4.56 | 67.1 | 76.8 | 143.9 |
| 26 | 5.01 | 74.3 | 86.1 | 160.4 |
| 28 | 5.49 | 82.0 | 96.0 | 178.0 |
| 30 | 5.98 | 89.9 | 106.6 | 196.5 |
| 32 | 6.49 | 98.3 | 117.8 | 216.1 |
| 34 | 7.01 | 107.0 | 129.6 | 236.6 |
| 36 | 7.54 | 116.1 | 142.2 | 258.3 |
| 38 | 8.05 | 125.6 | 155.4 | 281.0 |
| 40 | 8.68 | 135.5 | 169.4 | 304.9 |
| 42 | 9.26 | 145.8 | 184.2 | 330.0 |
| 44 | 9.86 | 156.5 | 199.7 | 356.2 |

N = 1/24 H + 7½"; K = 2N + 6"
Example: If H = 10', N = 12½", K = 2'-7",
A = 3'-7", B = 5'-8", C = 12'-3½", D = 13'-5½",
E = 15'-1½", F = 16'-3½", G = 14'-1", J = 15'-3"
Concrete; 1:2½:5, Class 2 D.

FIG. 12. STANDARD MASONRY PIERS. MICHIGAN STATE HIGHWAY COMMISSION.

**STEEL TUBULAR PIERS.**—Steel tubular piers are made of steel plates riveted together and filled with concrete. Where the piers are founded in soft material, piles are driven in the bottom of the tube, the piles being sawed off below the water line. The piles should extend at least two diameters of the tube above the bottom. The tubes are braced transversely by means of struts and tension diagonals above high water, and by diaphragm bracing below high water. Where the piers will be subject to blows from floating drift or logs, they should be protected by a timber crib-work or other device.

**Cooper's Standards.**—The tubular piers in Fig. 16 are from Cooper's "General Specifications for Foundations and Substructures for Highway and Electric Railway Bridges." Cooper specifies a minimum thickness of ⅜ in. for plates below and ¼ in. above the high water. The minimum sizes of tubular piers are as given in Fig. 16.

A steel tubular pier with a timber crib protection is given in Fig. 16. The crib is filled with loose rock.

A steel oblong pier, as designed by Cooper, is given in Fig. 17. The center of the truss is to come $a/2$ + one foot from the end of the pier. The width $a$, as specified by Cooper, is given in Fig. 17.

**American Bridge Company Standards.**—The American Bridge Company's standard tubular piers are shown in Fig. 18. The minimum diameters for a height of 15 feet to carry a single span and data on piers, pier beams and pier bracing are given in Fig. 18. In calculating the weight of a pier add one foot to the length of each tube. The weight of the concrete in two tubes is given in Fig. 18. The concrete is assumed to fill the tube and the space occupied by the piles should be deducted. The number of piles required for different diameters of tubes is given.

The number of piles required for large tubes agrees quite closely with Cooper's Specifications, but the number for small tubes is very much less.

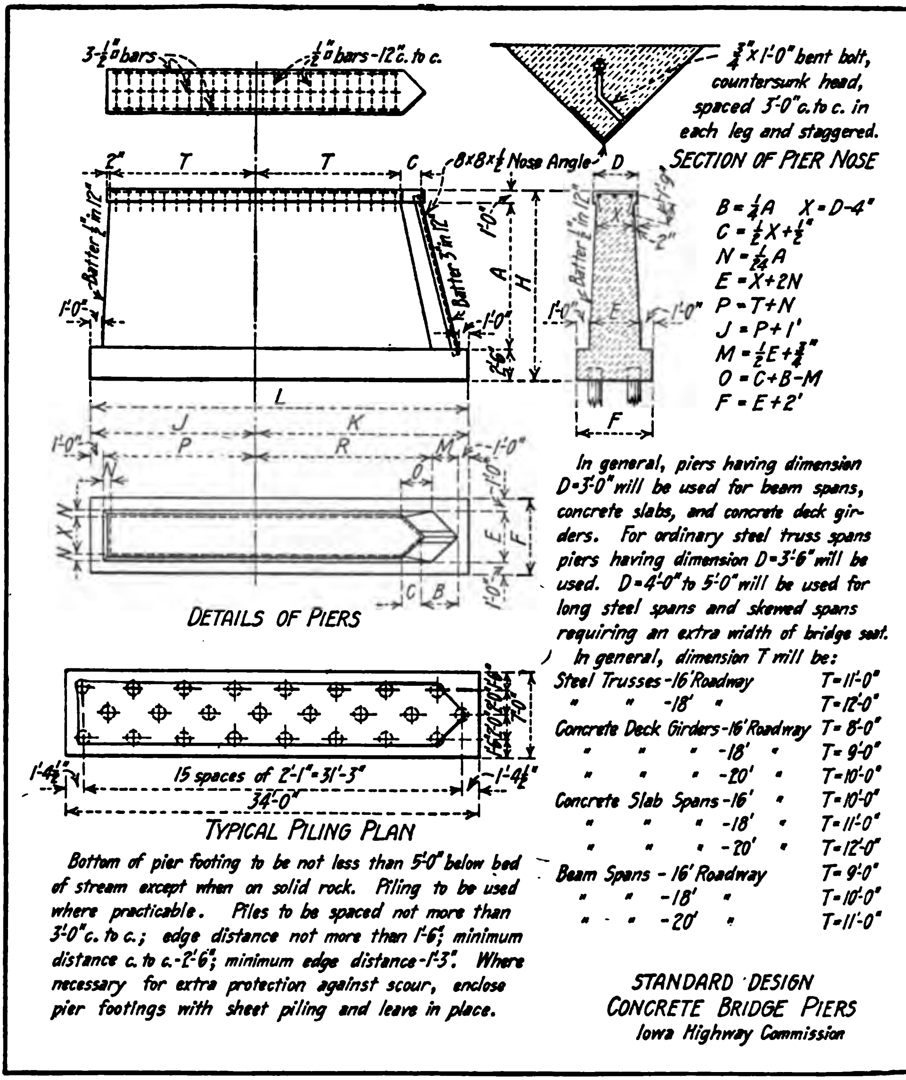



FIG. 13. STANDARD MASONRY PIERS. IOWA HIGHWAY COMMISSION.

*Pier Beams.*—The size of pier beams required for different panel lengths and clear distance between tubes in feet, are given in Fig. 18. The pier beam should be assumed as one foot longer than the clear distance between the tubes, in calculating the weight of the beams.

*Pier Bracing.*—The pier bracing for piers supporting the ends of two spans are given in Fig. 18. If the spans are unequal in length, enter the table with one-half of the algebraic sum of the spans.

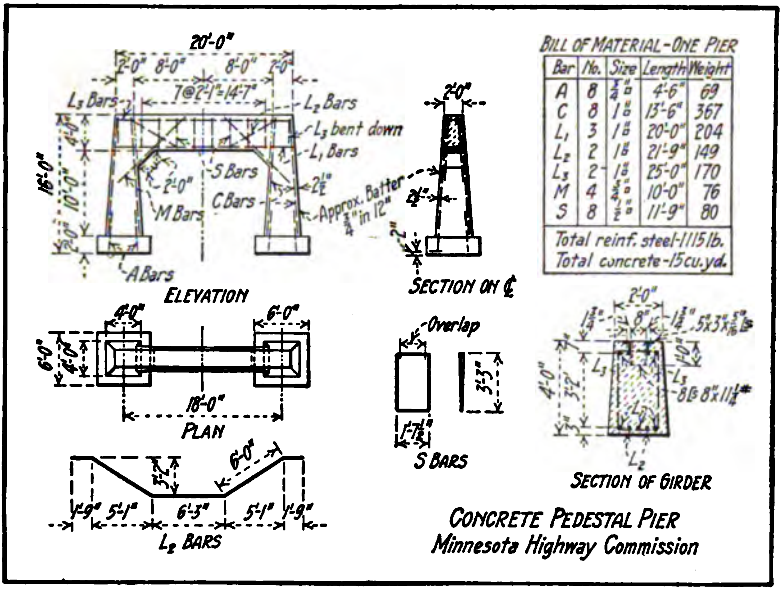



FIG. 14. CONCRETE PEDESTAL PIER. MINNESOTA HIGHWAY COMMISSION.

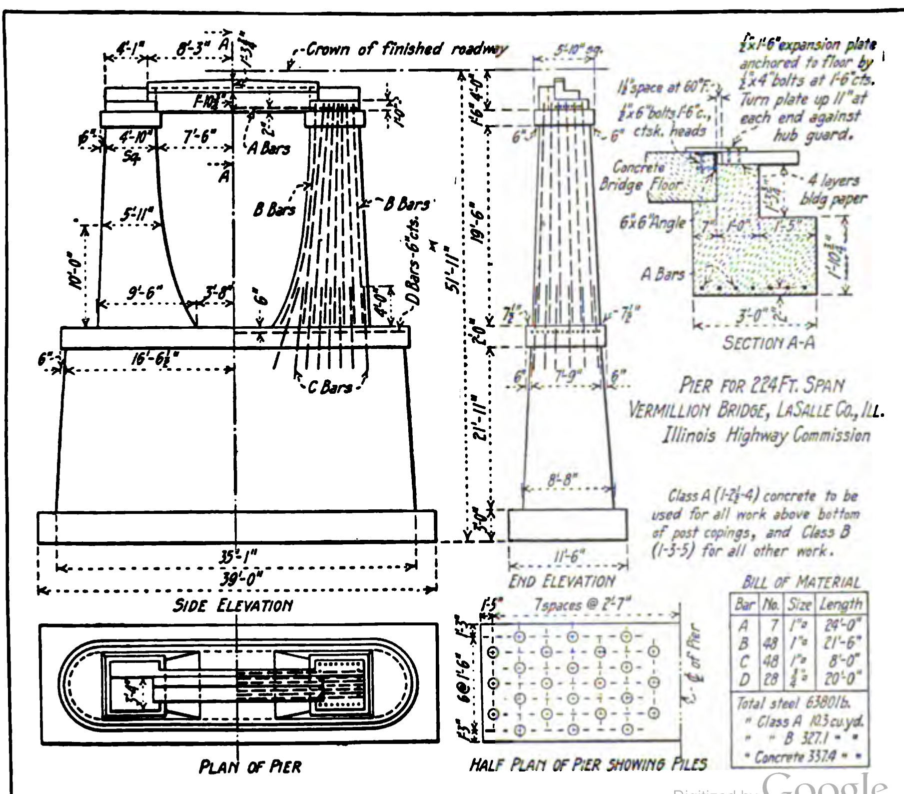
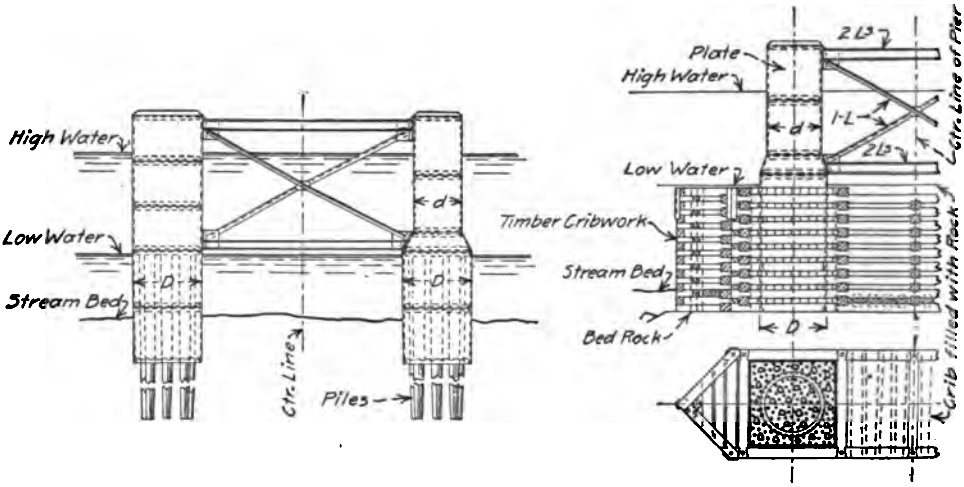



FIG. 15. CONCRETE PEDESTAL PIER, VERMILLION BRIDGE.

For example, for a pier carrying a 75-ft. and a 125-ft. span, enter the diagram with a span of 100 ft. Steel tubular piers should never be used for end abutments carrying a fill.

In calculating the weight of the diagonal bars the length of the bar should be multiplied by the weight per foot as obtained from a handbook, and the details for one bar added to the product. In calculating the weight of the struts add one foot to the clear length.




(a) STEEL TUBULAR PIERS

(b) CRIB CONSTRUCTION FOR STEEL TUBULAR PIERS

*MINIMUM SIZES OF STEEL TUBULAR PIERS, COOPER'S STANDARDS*

| Span in Feet | Highway & Single Track Electric Railway | | | Double Track Electric Railway | | |
|---|---|---|---|---|---|---|
| | Minimum Top, d | Diameter Bot. D. | Number of Piles. | Minimum Top d | Diameter Bot D | Number of Piles |
| 50 | 2'10" | 3'4" | 4 | 3'4" | 4'4" | 8 |
| 75 | 3'4" | 3'9" | 5 | 3'10" | 5'6" | 10 |
| 100 | 3'8" | 4'2" | 6 | 4'6" | 6'0" | 10 |
| 125 | 4'0" | 4'7" | 8 | 4'10" | 6'4" | 12 |
| 150 | 4'4" | 5'0" | 9 | 5'2" | 7'0" | 12 |
| 175 | 4'8" | 5'6" | 10 | 5'6" | 7'6" | 15 |
| 200 | 5'0" | 5'10" | 11 | 5'10" | 8'0" | 15 |
| 250 | 5'6" | 6'4" | 12 | 6'4" | 9'0" | 19 |

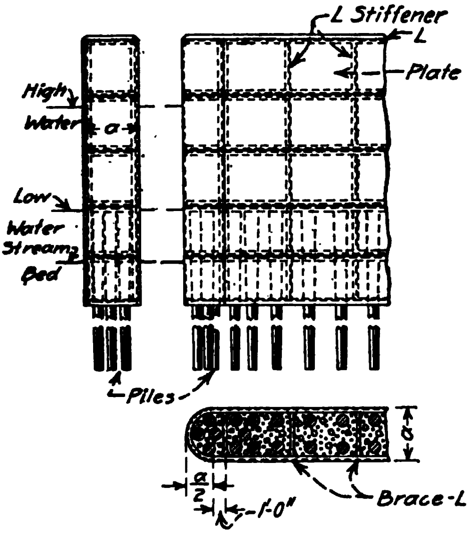
FIG. 16. STEEL TUBULAR PIERS FOR HIGHWAY AND ELECTRIC RAILWAY BRIDGES. COOPER'S STANDARDS.

*Pier Caps.*—Tubular piers may be capped with steel plate caps, may be finished with concrete, or may have a stone pedestal block. The weights given in Fig. 18 do not include the weights of steel caps.

**Specifications for Steel Tubular Piers for Highway and Electric Railway Bridges.**—The plates for the tubes shall be not less than ¼ in. thick for tubes up to 30 in. diameter, not less than $\frac{5}{16}$ in. for tubes from 30 to 48 in. in diameter, and not less than ⅜ in. for tubes from 48 to 72 in.

in diameter. Where the plates are in contact with the soil the thickness shall be increased at least $\frac{1}{16}$ in. For $\frac{5}{16}$-in. plate and less use $\frac{5}{8}$-in. rivets; for $\frac{3}{8}$-in. plate and over use $\frac{3}{4}$-in. rivets.

The horizontal seams shall be single lap joints riveted with a pitch of 4 diameters of rivet, while the vertical seams shall preferrably be butt riveted with single riveting spaced 4 diameters of rivet, up to 48 in. diameter of tubes, and double riveting with 3-in. spacing for tubes of larger diameter.




OBLONG STEEL PIERS

MINIMUM SIZES OF STEEL OBLONG PIERS
COOPER'S STANDARDS

| Span in Feet | Width a | |
|---|---|---|
| | Highway and Single Track Electric Railway | Double Track Electric Railway |
| 50 | 2'10" | 3'4" |
| 75 | 3'4" | 4'0" |
| 100 | 3'8" | 4'6" |
| 125 | 4'0" | 4'10" |
| 150 | 4'4" | 5'2" |
| 175 | 4'8" | 5'6" |
| 200 | 5'0" | 5'10" |
| 250 | 5'6" | 6'4" |

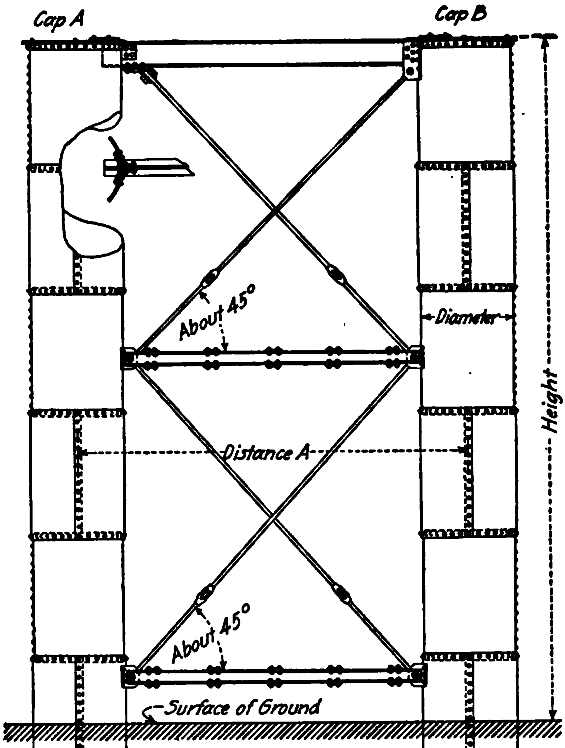
FIG. 17. STEEL OBLONG PIERS FOR HIGHWAY AND ELECTRIC RAILWAY BRIDGES. COOPER'S STANDARDS.

The bracing between cylinders shall be a solid web below high water level. Above high water level the bracing may consist of struts and diagonal rods. The diagonal rods in open bracing shall be inclined at an angle with the vertical of not less than 45 degrees. The rods shall be upset and be provided with turn-buckles. The bracing shall be made sufficiently strong to maintain the cylinders in an upright position when acted upon by the prescribed lateral wind loads without assistance of piles.

Piers 30 in. or less in diameter shall have one pile, and one additional pile shall be added for each increase of six (6) inches in diameter of cylinder. A cylinder 72 in. in diameter will then have eight (8) piles.

The materials and workmanship shall comply with the specifications for the highway bridge superstructure.

*Erection.*—Where the bottom will permit, the tubes shall be sunk well below possible scour by loading the tube, and excavating the material from the inside. For this purpose a clamshell bucket is very effective. Driving the tube with a pile driver will cut off the rivets in the horizontal seams and will not be permitted. After the tube is sunk, piles are to be driven inside of the steel shell, as closely together as possible, using care to get no pile nearer than 4 to 6 in. to the steel shell. The piles shall be driven to a good refusal, and the tops sawed off below the low water mark and reaching at least 2 diameters of the tube above the bottom. The space inside the tubes shall then be filled with concrete well tamped. Concrete shall not be deposited in running water if possible to prevent it.




MINIMUM DIAMETER OF STEEL TUBULAR PIERS FOR A HEIGHT OF 15 FEET TO CARRY A SINGLE SPAN

| Span Feet | Diameter Inches |
|---|---|
| 25 | 18 |
| 50 | 21 |
| 75 | 24 |
| 100 | 27 |
| 125 | 30 |
| 150 | 33 |
| 175 | 36 |
| 200 | 42 |

Increase diameter 3" for each additional 5 feet in height.

STEEL TUBULAR PIERS
AMERICAN BRIDGE COMPANY STANDARDS

CYLINDER PIERS

| All quantities for One Pair of Tubes | | | | | | | | Piles |
|---|---|---|---|---|---|---|---|---|
| Diam. of Tube | Weight per Vert. Ft. of 2 Tubes | | | | | | Cu. Yd. per Vert. Ft. | No. for One Tube |
| | 3/16" | 1/4" | 5/16" | 3/8" | 7/16" | 1/2" | | |
| 15" | 75# | 97# | 119# | 142# | 164# | 187# | 0·091 | 1 |
| 18 | 88 | 114 | 140 | 167 | 194 | 220 | 0·131 | 1 |
| 21 | 102 | 131 | 162 | 194 | 223 | 253 | 0·178 | 1 |
| 24 | 117 | 150 | 185 | 221 | 255 | 290 | 0·232 | 1 |
| 27 | 130 | 167 | 206 | 247 | 284 | 324 | 0·296 | 1 |
| 30 | 143 | 185 | 227 | 271 | 315 | 357 | 0·364 | 1 |
| 33 | 157 | 203 | 250 | 300 | 347 | 393 | 0·440 | 1 |
| 36 | 172 | 222 | 273 | 326 | 377 | 429 | 0·524 | 2 |
| 39 | 185 | 240 | 293 | 352 | 408 | 463 | 0·614 | 2 |
| 42 | 200 | 257 | 316 | 378 | 437 | 497 | 0·712 | 3 |
| 45 | 213 | 275 | 339 | 405 | 469 | 532 | 0·820 | 3 |
| 48 | 227 | 293 | 362 | 412 | 500 | 568 | 0·930 | 4 |
| 54 | | 329 | 405 | 485 | 563 | 636 | 1·178 | 5 |
| 60 | | 365 | 449 | 539 | 621 | 705 | 1·454 | 6 |
| 66 | | | 495 | 593 | 684 | 780 | 1·758 | 7 |
| 72 | | | 538 | 643 | 743 | 845 | 2·094 | 8 |
| 78 | | | | 698 | 805 | 917 | 2·458 | 10 |
| 84 | | | | 749 | 866 | 984 | 2·850 | 13 |

PIER BEAMS
FOR VARIOUS PANEL LENGTHS
AND CLEARANCES BETWEEN BEAMS.

| Span Length | Clearances for Various Sizes of I Beams | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | 8" I 18# | 9" I 21# | 10" I 25# | 12" I 31½# | 12" I 40# | 15" I 42# | 15" I 50# | 18" I 55# |
| 12'0" | 9'6" | 10'9" | 12'3" | 15'0" | 16'9" | 19'3" | 20'0" | 23'6" |
| 13-0 | 9-0 | 10-6 | 11-9 | 14-3 | 16-0 | 18-6 | 19-3 | 22-6 |
| 14-0 | | 10-0 | 11-3 | 13-9 | 15-6 | 17-9 | 18-6 | 21-9 |
| 15-0 | | 9-9 | 11-0 | 13-3 | 15-0 | 17-0 | 18-0 | 21-0 |
| 16-0 | | 9-6 | 10-9 | 13-0 | 14-6 | 16-6 | 17-3 | 20-3 |
| 17-0 | | 9-0 | 10-3 | 12-6 | 14-0 | 16-0 | 16-9 | 19-9 |
| 18-0 | | | 10-0 | 12-3 | 13-6 | 15-6 | 16-3 | 19-3 |
| 19-0 | | | 9-9 | 12-0 | 13-3 | 15-3 | 16-0 | 18-9 |
| 20-0 | | | 9-6 | 11-6 | 13-0 | 14-9 | 15-6 | 18-3 |
| 21-0 | | | 9-3 | 11-3 | 12-6 | 14-6 | 15-3 | 17-9 |

PIER BRACING

| Supported Distance | Size, Wt. per Ft. and Details 1 Rod | STRUTS:—Sizes & Wts. per Ft. for Various Roadways | | | | |
|---|---|---|---|---|---|---|
| | | 12'0" | 14'0" | 16'0" | 18'0" | 20'0" |
| 25' | 7/8"□ @ 2·6#/Ft. Details, 1-Rod 20# | 2[4"×5¼# 17#/Ft. | 2[4"×5¼# 17#/Ft. | 2[4"×5¼# 17#/Ft. | 2[4"×5¼# 17#/Ft. | 2[4"×5¼# 17#/Ft. |
| 50' | 1⅛"□ @ 4·3#/Ft. Details, 1-Rod 30# | 2[4"×5¼# 17#/Ft. | 2[4"×5¼# 17#/Ft. | 2[4"×5¼# 17#/Ft. | 2[4"×5¼# 17#/Ft. | 2[5"×6½# 19#/Ft. |
| 75' | 1⅜"□ @ 6·4#/Ft. Details, 1-Rod 45# | 2[4"×5¼# 17#/Ft. | 2[4"×5¼# 17#/Ft. | 2[5"×6½# 19#/Ft. | 2[5"×6½# 19#/Ft. | 2[6"×8# 22#/Ft. |
| 100' | 1⅝"□ @ 9·0#/Ft. Details, 1-Rod 65# | 2[5"×6½# 19#/Ft. | 2[5"×6½# 19#/Ft. | 2[6"×8# 22#/Ft. | 2[6"×8# 22#/Ft. | 2[7"×9¾# 26#/Ft. |
| 125' | 1⅞"□ @ 12·0#/Ft. Details, 1-Rod 90# | 2[6"×8# 22#/Ft. | 2[6"×8# 22#/Ft. | 2[6"×8# 22#/Ft. | 2[7"×9¾# 26#/Ft. | 2[7"×9¾# 26#/Ft. |

FIG. 18. STEEL TUBULAR PIERS FOR HIGHWAY BRIDGES. AMERICAN BRIDGE COMPANY.

Where piers are founded on rock, the tubes are to be anchored to the rock and then filled with concrete. Or cribs may be sunk on the rock and the tube set in a pocket in the crib and resting on the rock. The space outside the tube is then filled with concrete and the tube is filled with concrete in the usual manner.

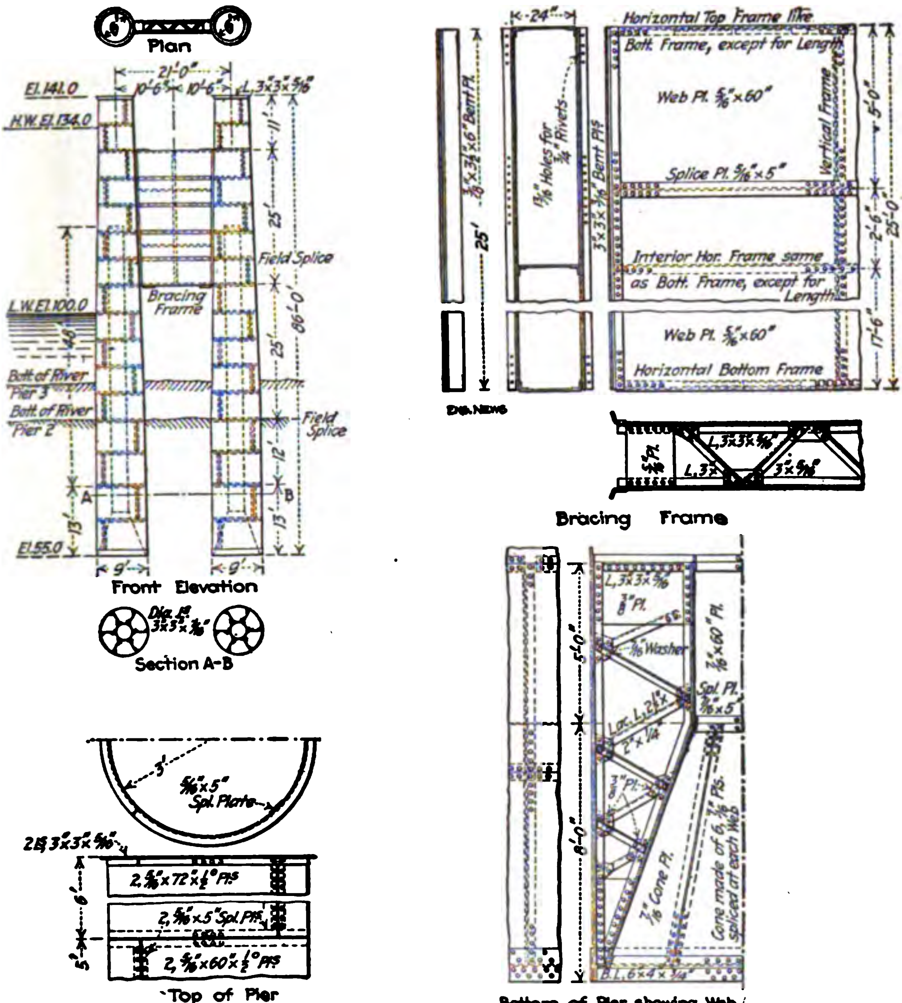



FIG. 19. STEEL TUBULAR PIERS, TRAIL, B. C.

**Cylinder Piers for Highway Bridge, Trail B. C.***—Steel cylinder piers, Fig. 19, were used for a steel highway bridge designed by Waddell and Harrington, consulting engineers, and built across the Columbia River at Trail, B. C. The main spans are 172 ft. 8 in. long and are carried on piers made of two steel cylinders filled with concrete. The steel cylinders are 9 ft. in diameter

* Engineering News, Dec. 5, 1912.

at the bottom and 6 ft. in diameter at the top, and are 86 ft. long. The cylinders are made of plates $\frac{1}{2}$ in. thick and are connected by a double plate web diaphragm, each diaphragm made of $\frac{5}{16}$-in. plates spaced 24 in. apart and 25 ft. high, and reaching from below low water to above high water. The diaphragms were covered and filled with concrete. The cylinders are spaced 21 ft. centers. The piers were sunk by the pneumatic process.

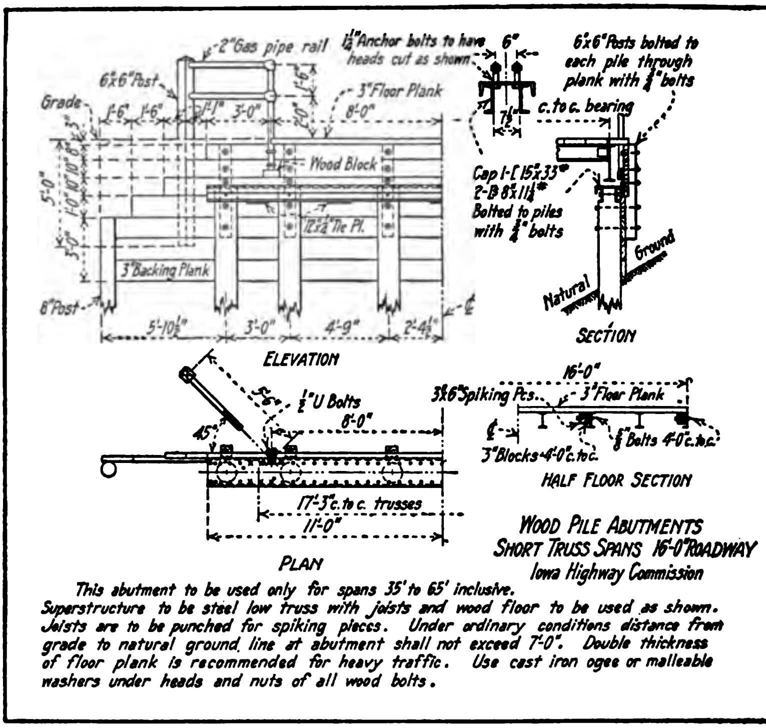



FIG. 20. TIMBER PILE ABUTMENT FOR STEEL HIGHWAY BRIDGES. IOWA HIGHWAY COMMISSION.

**Timber Abutment.**—The details of a timber abutment for steel low truss highway bridges, as designed by the Iowa Highway Commission are shown in Fig. 20. This abutment is to be used for steel low truss bridges with timber floor with spans of 35 ft. to 65 ft. This abutment is especially suited to crossings of drainage ditches where it is difficult to get satisfactory foundations for masonry abutments.

Specifications for abutments and piers are given in Appendix II.

**Reference.**—For details of abutments and piers for railway bridges, see the author's "Structural Engineers' Handbook."

# CHAPTER XXI.

## Design of Reinforced Concrete Bridges.

**Introduction.**—The design of slab and girder bridges will be discussed in this chapter. The design of culverts will be considered in Chapter XXII, and the design of arches in Chapter XXIII.

**SLAB BRIDGES.**—The distribution of concentrated loads on reinforced concrete slabs has been considered in Chapter IX. The floor of a slab bridge is designed as any simply supported slab for a span length equal to the span length of the bridge. Shear in the slab should be investigated and stirrups and bent-up bars should be provided where necessary. Both the straight longitudinal and bent-up bars should be hooked at the ends. The slab bridge has the advantage that it is simple in design, and requires less material and labor in building the forms and less labor

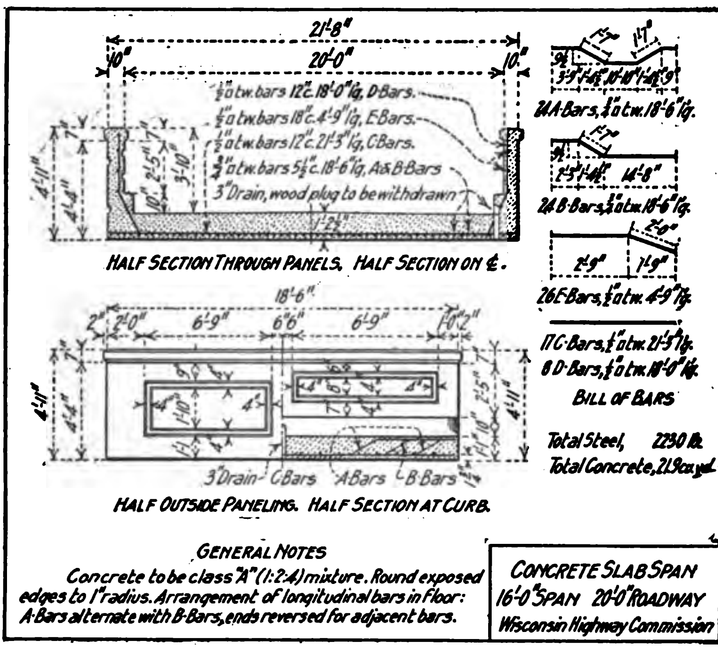



Fig. 1. Reinforced Concrete Slab Bridge. Wisconsin Highway Commission.

in placing the reinforcement than any other type of concrete bridge. It also admits of widening the roadway without impairing the strength of the existing structure. It has the disadvantage that it is not economical of materials for spans of more than about 20 ft. Details of a reinforced concrete slab bridge having a 16-ft. span and a 20-ft. roadway are shown in Fig. 1. This bridge was designed to carry a 15-ton road roller. This bridge is designed to be carried on independent abutments. Quantities of concrete and steel in standard slab bridges designed by the Wisconsin Highway Commission are given in Table I.

### TABLE I.

QUANTITIES OF CONCRETE AND STEEL IN REINFORCED SLAB BRIDGES.
WISCONSIN HIGHWAY COMMISSION.

| Span, Ft. | 16-Ft. Roadway. | | 18-Ft. Roadway. | | 20-Ft. Roadway. | | 24-Ft. Roadway. | |
|---|---|---|---|---|---|---|---|---|
| | Lb. Steel. | Cu. Yd. Concrete. | Lb. Steel. | Cu. Yd. Concrete. | Lb. Steel. | Cu. Yd. Concrete. | Lb. Steel. | Cu. Yd. Concrete. |
| 6 | 480 | 5.7 | 520 | 6.1 | 570 | 6.4 | 660 | 7.1 |
| 8 | 700 | 7.7 | 760 | 8.3 | 830 | 8.8 | 960 | 9.9 |
| 10 | 930 | 9.9 | 1,010 | 10.6 | 1,090 | 11.3 | 1,290 | 12.6 |
| 12 | 1,200 | 12.6 | 1,310 | 13.6 | 1,430 | 14.6 | 1,700 | 16.4 |
| 14 | 1,520 | 15.8 | 1,660 | 17.0 | 1,810 | 18.3 | 2,130 | 20.8 |
| 16 | 1,890 | 19.3 | 2,060 | 20.8 | 2,230 | 22.4 | 2,600 | 25.4 |
| 18 | 2,180 | 22.7 | 2,450 | 24.5 | 2,640 | 26.4 | 3,120 | 29.9 |
| 20 | 2,710 | 27.0 | 2,960 | 29.4 | 3,220 | 31.7 | 3,780 | 36.1 |
| 22 | 3,280 | 32.2 | 3,610 | 34.9 | 3,950 | 37.9 | 4,620 | 43.3 |
| 24 | 4,190 | 37.8 | 4,610 | 41.2 | 5,040 | 44.5 | 5,890 | 50.9 |

Details of a reinforced concrete slab bridge resting on reinforced concrete abutments are shown in Fig. 2. The slab is designed to support the tops of the abutments and to take the thrust of the filling. As the bridge is designed it will act as a simple span resting on the tops of the abutments. The Wisconsin Highway Commission has prepared standard plans for this type of bridge for spans of 8 ft. to 24 ft., and for roadways of 16 ft., 18 ft., 20 ft., and 24 ft. Data on spans with 20-ft. and 24-ft. roadways are given in Table II.

### TABLE II.

DATA ON CONCRETE SLAB BRIDGES ON REINFORCED CONCRETE ABUTMENTS.
WISCONSIN HIGHWAY COMMISSION.

| 20-Ft. Roadway. | | | | | | 24-Ft. Roadway. | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Span, Ft. | Depth Slab. In. | Thickness Abutment Slab, In. | Concrete, Cu. Yd. | | Reinforcing Steel, Lb. | Span, Ft. | Depth Slab, In. | Thickness Abutment Slab, In. | Concrete, Cu. Yd. | | Reinforcing Steel, Lb. |
| | | | Span. | Abutments. | | | | | Span. | Abutments. | |
| 8 | 8 | 12 | 8.8 | 33.5 | 1,910 | 6 | 7 | 12 | 7.1 | 30.1 | 1,590 |
| 12 | 10½ | 12 | 14.6 | 41.9 | 2,720 | 14 | 12 | 12 | 20.8 | 45.5 | 3,560 |
| 16 | 13 | 15 | 22.4 | 46.2 | 3,540 | 20 | 15½ | 18 | 36.1 | 55.1 | 5,240 |
| 22 | 17 | 21 | 37.9 | 54.3 | 5,370 | 24 | 19 | 21 | 50.9 | 60.1 | 7,360 |

Abutment slab 5 ft. above footing in 6-ft. span, 6 ft. above footing in 8-ft. span and 7 ft. above footing in other spans. For details of a bridge with 16-ft. span and 20-ft. roadway, see Fig. 2.

Details of a standard concrete slab bridge designed by the Iowa Highway Commission are given in Fig. 3. These bridges are designed with an 18-ft. roadway, and with spans of 14 ft. to 24 ft. Data for the design and quantities of concrete and steel are given in Fig. 3. Three layers of tar paper are put between the slab and the abutment to make an expansion joint.

Standard reinforced concrete bridges of the Iowa Highway Commission are designed for a 15-ton traction engine with two-thirds of total load on rear axle, or 100 lb. per sq. ft. of roadway and sidewalks. Each rear wheel is assumed as distributed 6 ft. transversely and 5 ft. longitudinally. For thin slabs on girders the same wheel load is assumed as distributed 4 ft. transversely and 4 ft. longitudinally. Allowable stresses in lb. per sq. in. are: compression in concrete, 600; shear in concrete, 100 (as a measure of diagonal tension); tension in steel 16,000.

For design of a slab bridge see Fig. 19.

Data for floor slabs designed by the Ohio State Highway Department are given in Table III.

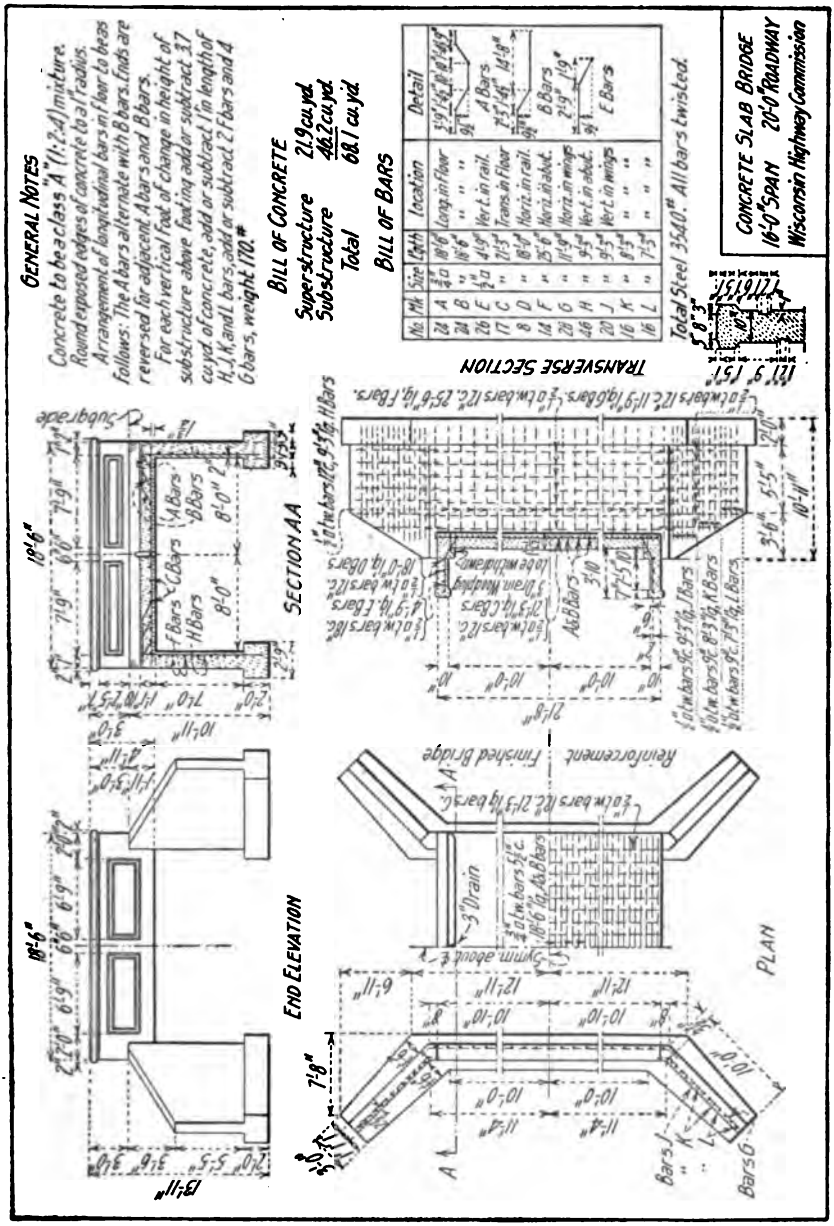
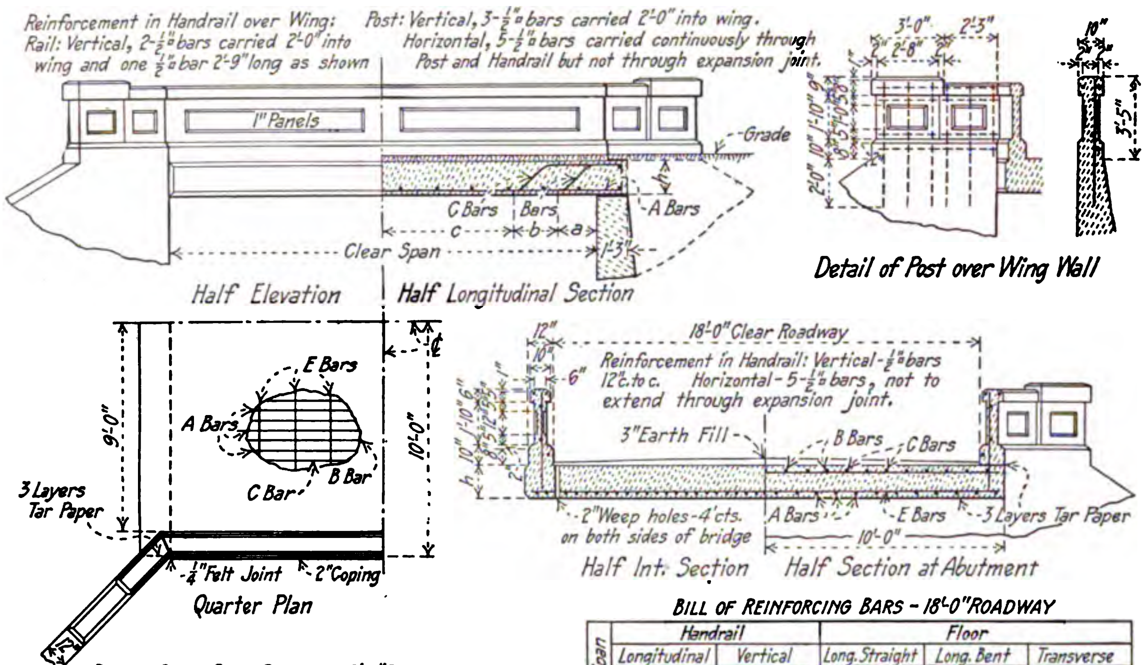



FIG. 2. REINFORCED CONCRETE SLAB BRIDGE ON REINFORCED CONCRETE ABUTMENTS. WISCONSIN HIGHWAY COMMISSION.




**DESIGN DATA - SLAB BRIDGES - 18'-0" ROADWAY**

| Span | Dimensions | | | | Longitud. Bars | | Transverse Bars | | Est. Quantities 18'-0" Roadway | | Quantities for 2' change of Rdwy. | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | h | a | b | c | Size | c. to c. | Size | c. to c. | Concrete | Steel | Concrete | Steel |
| 14 | 13½ | 1-1 | 1-5 | 4-6 | ¾" □ | 6½" | ½" □ | 1'-6" | 16.0 cu.yd. | 1635# | 1.37 cu.yd. | 141# |
| 16 | 15 | 1-4 | 1-8 | 4-6 | ¾" □ | 5½" | ½" □ | 1'-6" | 19.8 " " | 2055# | 1.71 " " | 180# |
| 18 | 16½ | 1-8 | 1-10 | 5-6 | ¾" □ | 5" | ½" □ | 1'-6" | 23.7 " " | 2430# | 2.08 " " | 215# |
| 20 | 18½ | 1-8 | 1-10 | 6-6 | 1" □ | 8" | ½" □ | 1'-6" | 29.0 " " | 2955# | 2.57 " " | 264# |
| 22 | 20½ | 1-9 | 2-3 | 7-0 | 1" □ | 7" | ½" □ | 1'-6" | 34.7 " " | 3550# | 3.10 " " | 321# |
| 24 | 22 | 2-0 | 2-6 | 7-6 | 1" □ | 6½" | ½" □ | 1'-6" | 40.0 " " | 4125# | 3.59 " " | 375# |

**BILL OF REINFORCING BARS - 18'-0" ROADWAY**

| Span | Handrail | | | | | | Floor | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Longitudinal | | | Vertical | | | Long. Straight | | | Long. Bent | | | Transverse | | |
| | No. | Size | Length | No. | Size | Length | No. | Size | Length | No. | Size | Length | No. | Size | Length |
| 14 | 10 | ½" □ | 14'-0" | 30 | ½" □ | 4'-0" | 19 | ¾" □ | 16'-6" | 18 | ¾" □ | 17'-2" | 13 | ½" □ | 20'-0" |
| 16 | 10 | ½" □ | 16'-0" | 34 | ½" □ | 4'-0" | 22 | ¾" □ | 18'-6" | 21 | ¾" □ | 19'-3" | 15 | ½" □ | 20'-0" |
| 18 | 10 | ½" □ | 18'-0" | 38 | ½" □ | 4'-0" | 24 | ¾" □ | 20'-6" | 23 | ¾" □ | 21'-4" | 16 | ½" □ | 20'-0" |
| 20 | 10 | ½" □ | 20'-0" | 42 | ½" □ | 4'-0" | 15 | 1" □ | 22'-6" | 15 | 1" □ | 23'-6" | 17 | ½" □ | 20'-0" |
| 22 | 10 | ½" □ | 22'-0" | 46 | ½" □ | 4'-0" | 17 | 1" □ | 24'-6" | 17 | 1" □ | 25'-8" | 18 | ½" □ | 20'-0" |
| 24 | 10 | ½" □ | 24'-0" | 50 | ½" □ | 4'-0" | 19 | 1" □ | 26'-6" | 18 | 1" □ | 27'-9" | 20 | ½" □ | 20'-0" |

Note: The quantities shown in tables do not include handrail over wings, which add 1.9 cu.yd. of concrete and 180 lb. of steel per bridge.

STANDARD DESIGN FOR CONCRETE SLAB BRIDGES
Iowa Highway Commission

FIG. 3. REINFORCED CONCRETE SLAB BRIDGE. IOWA HIGHWAY COMMISSION.

## TABLE III.

**Thickness of Concrete Floor Slabs, Used with Joists. Ohio State Highway Department.**

| Span, Ft. | Weight of Concentrated Load. 10-Ton Truck. Thickness, In. | 10-Ton Truck. Reinforcing. Sq. Bar, In. | 10-Ton Truck. Reinforcing. Spacing, In. | 15-Ton Truck. Thickness, In. | 15-Ton Truck. Reinforcing. Sq. Bar. In. | 15-Ton Truck. Reinforcing. Spacing, In. | 20-Ton Truck. Thickness, In. | 20-Ton Truck. Reinforcing. Sq. Bar, In. | 20-Ton Truck. Reinforcing. Spacing, In. |
|---|---|---|---|---|---|---|---|---|---|
| 3 | 5 | ½ | 7½ | 5½ | ½ | 5¾ | 6½ | ½ | 5¼ |
| 4 | 5½ | ½ | 7 | 6 | ½ | 5¼ | 7 | ⅝ | 4¾ |
| 5 | 5½ | ½ | 6 | 6½ | ½ | 5¼ | 7 | ⅝ | 6¼ |
| 6 | 6 | ½ | 6 | 7 | ⅝ | 7¾ | 7½ | ⅝ | 6½ |
| 7 | 6 | ½ | 5¼ | 7 | ⅝ | 7 | 8 | ⅝ | 6½ |
| 8 | 6½ | ⅝ | 7½ | 7½ | ⅝ | 6½ | 8½ | ⅝ | 5¾ |
| 9 | 7 | ⅝ | 6¾ | 8 | ⅝ | 6 | 9 | ⅝ | 5¼ |
| 10 | 7½ | ⅝ | 6½ | 8½ | ⅝ | 5½ | 9½ | ⅝ | 5 |
| 11 | 8 | ⅝ | 6 | 9 | ⅝ | 5¼ | 10 | ¾ | 6¾ |
| 12 | 8½ | ⅝ | 5¾ | 9½ | ¾ | 7 | 10½ | ¾ | 6½ |
| 14 | 9½ | ¾ | 7¼ | 10½ | ¾ | 6½ | 11½ | ¾ | 5¾ |
| 16 | 10½ | ¾ | 6½ | 11½ | ¾ | 5¾ | 12½ | ¾ | 5¼ |
| 18 | 11½ | ¾ | 6 | 12½ | ¾ | 5¼ | 13½ | ⅞ | 6¾ |
| 20 | 12½ | ¾ | 5¼ | 13½ | ⅞ | 6½ | 14½ | ⅞ | 6 |

Floor covering 65 lb. per sq. ft. Reinforcement lower side. Center of reinforcing 1 in. from face of slab. Impact 16⅔ per cent. Allowable tension in steel 16,000 lb. per sq. in. Compression in concrete 750 lb. per sq. in.

**T-BEAM BRIDGES.**—Details of a reinforced concrete T-beam bridge with a 30-ft. span, as designed by the Ohio State Highway Department are given in Fig. 4. Data are given for 16-ft. and 20-ft. roadways. Tar felt is used for expansion joints.

Details of a reinforced concrete T-beam bridge, with 28-ft. span and 18-ft. roadway as designed by the Michigan State Highway Department are given in Fig. 5. This bridge is designed to carry an 18-ton road-roller.

Data and quantities for reinforced concrete T-beam bridges designed by the Massachusetts Highway Commission are given in Table IV.

The bridges were designed for a 20-ton motor truck with axles spaced 12 ft. and 6-ft. gage, with 14 tons on rear axle and 6 tons on front axle, and uniform load of 100 lb. per sq. ft. Bridges have a clear roadway of 23′ 6″ and a distance out to out of 27′ 8″, with allowable stresses as given in Report Joint Committee, Am. Soc. C. E., 1913. Stirrups ⅜ in. sq. spaced 6 in. to 8 in. in end quarters, and 12 in. to 18 in. in middle half were used in beams 10-ft. to 28-ft. span; stirrups ½ in. sq. spaced 8 in. to 10 in. in end quarters; and 16 in. to 24 in. in middle half in beams of 34-ft. to 40-ft. span. One-half of horizontal bars, or one-half of number of horizontal bars less one, are bent-up, not more than two bars being bent up at one place. Points of bending up are about one-eighth of span apart. Bent-up longitudinal bars are anchored at the ends of the beams by means of hooks bent through 180 degrees.

Details of standard reinforced concrete T-beam bridges designed by U. S. Bureau of Public Roads are given in Fig. 6 and Fig. 7. The plans in Fig. 6 are for spans from 16 ft. to 26 ft., inclusive, and a 20-ft. roadway; while plans in Fig. 7 are for spans from 28 ft. to 40 ft., inclusive, and a 20-ft. roadway. Dimensions, data and quantities for the different spans are given in detail. These girder bridges are designed for a 15-ton truck, with axles spaced 10 ft. centers, and

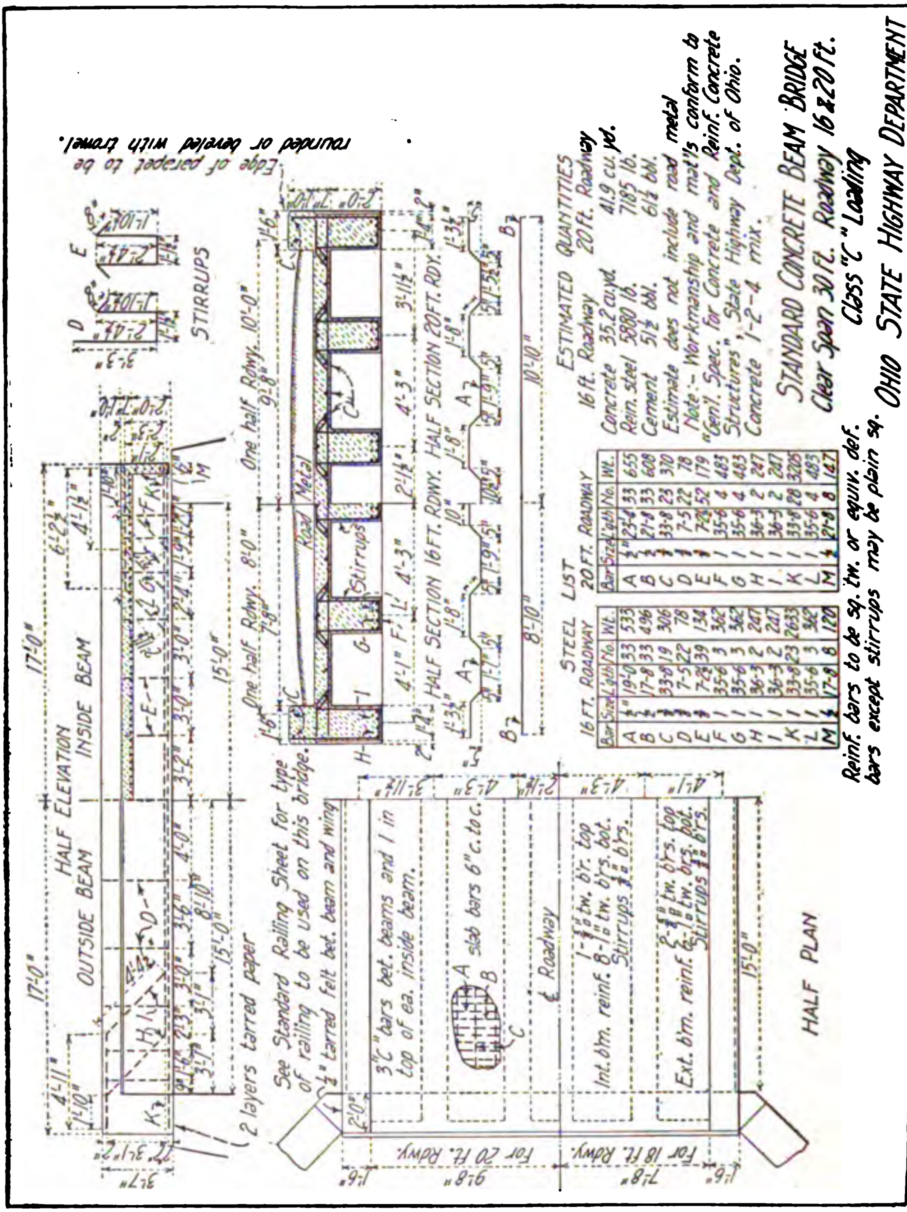



FIG. 4. REINFORCED CONCRETE T-BEAM BRIDGE. OHIO STATE HIGHWAY DEPARTMENT.

wheels spaced 6 ft. centers. Two-thirds of the total load is carried on the rear axle. An impact allowance of 30 per cent is provided for. Concrete is 1 : 2 : 4 mixture with agregate 1½ in. maximum. Allowable tension in steel reinforcement, 16,000 lb. per sq. in.; allowable compression in concrete, 650 lb. per sq. in.; allowable shear on concrete, 110 lb. per sq. in.

## TABLE IVa.

QUANTITIES IN CONCRETE DECK GIRDER HIGHWAY BRIDGES. U. S. BUREAU OF PUBLIC ROADS.

| Span, Ft. | 16-Ft. Roadway. | | 18-Ft. Roadway. | | 20-Ft. Roadway. | | 24-Ft. Roadway. | |
|---|---|---|---|---|---|---|---|---|
| | Concrete, Cu. Yd. | Steel, Lb. | Concrete, Cu. Yd. | Steel, Lb. | Concrete, Cu. Yd. | Steel, Lb. | Concrete, Cu. Yd. | Steel, Lb. |
| 16 | 14.3 | 2,820 | 15.3 | 3,160 | 16.5 | 3,510 | 19.2 | 3,980 |
| 18 | 16.0 | 3,130 | 17.7 | 3,520 | 19.0 | 3,880 | 22.2 | 4,430 |
| 20 | 17.8 | 3,770 | 19.3 | 4,330 | 21.2 | 4,760 | 24.2 | 5,470 |
| 22 | 20.1 | 4,100 | 21.8 | 4,690 | 23.3 | 5,140 | 27.4 | 5,930 |
| 24 | 22.3 | 4,460 | 23.6 | 5,710 | 25.7 | 6,230 | 29.6 | 7,220 |
| 26 | 23.8 | 5,260 | 25.7 | 6,140 | 28.0 | 6,670 | 32.4 | 7,770 |
| 28 | 29.1 | 5,870 | 30.2 | 7,070 | 32.8 | 7,440 | 37.7 | 8,940 |
| 30 | 32.6 | 6,260 | 34.1 | 7,570 | 36.9 | 7,980 | 42.5 | 9,580 |
| 32 | 34.6 | 7,560 | 38.1 | 8,100 | 41.2 | 8,500 | 47.6 | 10,240 |
| 34 | 38.3 | 8,100 | 42.5 | 8,580 | 43.5 | 10,180 | 53.0 | 10,850 |
| 36 | 42.2 | 8,550 | 45.5 | 10,300 | 47.9 | 10,770 | 56.6 | 13,040 |
| 38 | 46.4 | 9,070 | 50.1 | 10,910 | 52.8 | 11,420 | 62.5 | 13,810 |
| 40 | 50.8 | 9,540 | 55.0 | 11,520 | 57.8 | 12,030 | 68.6 | 14,600 |

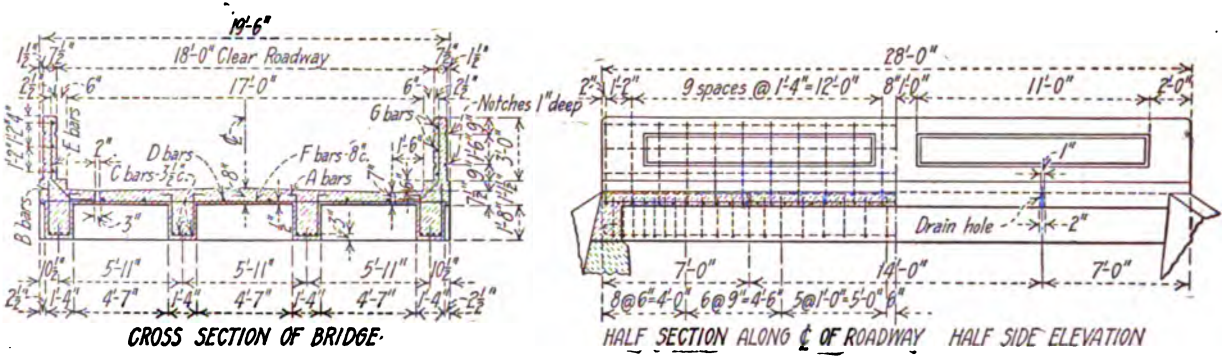
## TABLE IV.

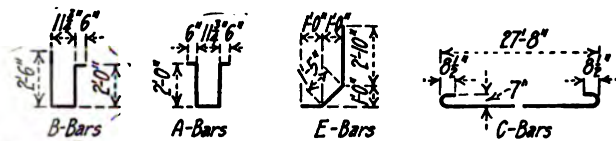
DATA AND QUANTITIES, FOR REINFORCED CONCRETE T-BEAM BRIDGES.
MASSACHUSETTS HIGHWAY COMMISSION.
Roadway 23 ft. 6 in. Designed for 20-ton Motor Truck.

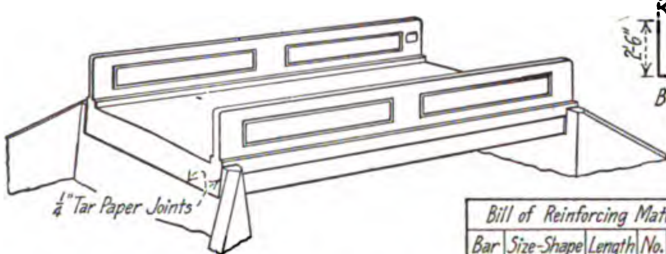
| Span, Ft. | Number T-Beams. | C. to C. T-Beams, In. | Depth T-Beam Below Floor Slab, In. | Width, T-Beam, In. | Depth Floor Slab, In. | Reinforcement. T-Beam Sq. Bars. No. | Reinforcement. T-Beam Sq. Bars. Size. | Reinforcement. Slab, Sq. Bars. Size. | Reinforcement. Slab, Sq. Bars. Spacing. | Concrete, Cu. Yd. | Reinforcing, Lb. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 10 | 8 | 44 | 13 | 10 | 5 | 3 | ⅞″ | ⅜″ | 6″ | 11.3 | 1,642 |
| 13 | 8 | 44 | 14½ | 10 | 5 | 3 | 1″ | ⅜″ | 6″ | 14.9 | 2,330 |
| 18 | 8 | 44 | 18 | 10 | 5 | 3 | 1⅛″ | ⅜″ | 6″ | 21.3 | 3,610 |
| 19 | 8 | 44 | 19½ | 10 | 5 | 5 | ⅞″ | ⅜″ | 6″ | 23.0 | 3,850 |
| 25 | 8 | 44 | 23 | 10 | 5 | 5 | 1″ | ⅜″ | 6″ | 32.1 | 5,772 |
| 25 | 7 | 53 | 22½ | 11 | 5 | 5 | 1⅛″ | ⅜″ | 6″ | 33.0 | 6,175 |
| 28 | 7 | 53 | 27½ | 11 | 5 | 5 | 1⅛″ | ⅜″ | 6″ | 39.5 | 6,993 |
| 34 | 7 | 53 | 33½ | 11 | 5 | 6 | 1⅛″ | ⅜″ | 6″ | 51.8 | 10,760 |
| 34 | 6 | 64 | 32½ | 14 | 6 | 6 | 1¼″ | ½″ | 6″ | 55.8 | 11,220 |
| 40 | 6 | 64 | 37½ | 14 | 6 | 7 | 1¼″ | ½″ | 6″ | 69.8 | 15,875 |
| *28 | 7 | 53 | 21 | 15 | 6 | 11 | ⅞″ | ⅜″ | 6″ | 42.4 | 8,610 |
| *34 | 7 | 53 | 26 | 15 | 6 | 10 | 1″ | ½″ | 6″ | 57.8 | 13,070 |
| *40 | 6 | 64 | 31 | 17 | 7 | 11 | 1⅛″ | ½″ | 6″ | 73.1 | 17,582 |

* Special Bridges.

The U. S. Bureau of Public Roads has also prepared standard plans for the same spans with 16-ft. roadway, 18-ft. roadway, and 24-ft. roadway. Quantities of concrete and reinforcing steel in concrete deck girders with 16-ft., 18-ft., 20-ft., and 24-ft. roadway are given in Table IVa. Deck concrete girders with 16-ft. roadway have 4 beams, with 18-ft. and 20-ft. roadway have 5 beams, with 24-ft. roadway have 6 beams.




Material and workmanship to be in accordance with Mich. State Highway Dept. specifications, 1916 edition. All steel to be medium, open-hearth steel. Reinforcement to be wired with No. 18 wire. Approx. 33 cu. yds. 1-2-4 concrete in bridge. Bridge designed to carry an 18-Ton road roller.

**Bill of Reinforcing Material**

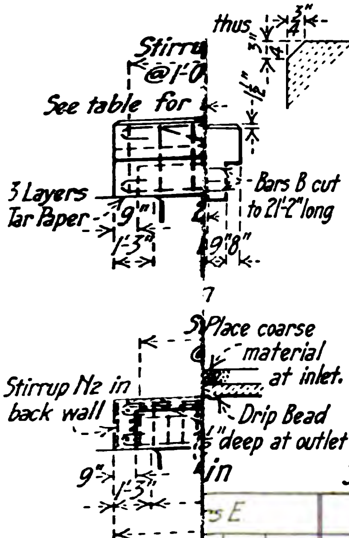
| Bar | Size-Shape | Length | No. | Weight |
|---|---|---|---|---|
| A | ½" Square | 6'-0" | 76 | 388 |
| B | ½" " | 6'-0" | 76 | 388 |
| C | 1¼" " | 29'-9" | 16 | 2528 |
| D | ¾" Round | 28'-0" | 6 | 252 |
| E | ¾" " | 5'-3" | 44 | 347 |
| F | ¾" " | 19'-3" | 41 | 1184 |
| G | ¾" " | 27'-6" | 6 | 247 |

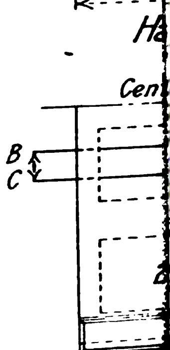
SURFACE TREATMENT OF CONCRETE ROADWAY

The surface of the concrete shall be thoroughly cleaned, and coal tar (heated to from 250° to 350° Fahr.) shall be evenly spread over the surface. Immediately after application of tar, and while the tar is still hot, the surface shall be evenly covered with ½" of clean, coarse, sharp sand. The application of tar must not be made when the surface of the concrete is damp.

CONCRETE T-BEAM BRIDGE
28'-0" SPAN 18'-0" ROADWAY
Michigan State Highway Dept.

FIG. 5. REINFORCED CONCRETE T-BEAM BRIDGE. MICHIGAN STATE HIGHWAY DEPARTMENT.



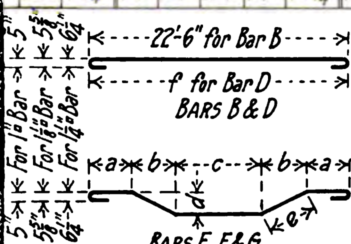




# DIMENSIONS AND QUANTITIES

| Clear Span Feet | Total Length Feet | Dimensions X | Dimensions Y | Dimensions Z | Stirrup Spacing: Stirrups M | Stirrup Spacing: Stirrups N1 | Stirrup Spacing: Stirrups N2 | Quantities: Concrete Cu yd. | Quantities: Steel Lb. |
|---|---|---|---|---|---|---|---|---|---|
| 16 | 18'-6" | 2'-11" | 1'-2" | 1'-4" | 3 @ 12"<br>9 @ 15"<br>3 @ 12" | 26 @ 8" | 10 @ 6"<br>9 @ 10"<br>10 @ 6" | 16.5 | 3510 |
| 18 | 20'-6" | 3'-1" | 1'-4" | 1'-6" | 4 @ 12"<br>9 @ 15"<br>4 @ 12" | 29 @ 8" | 11 @ 6"<br>10 @ 10"<br>11 @ 6" | 19.0 | 3880 |
| 20 | 22'-6" | 3'-2" | 1'-5" | 1'-7" | 4 @ 12"<br>10 @ 15"<br>4 @ 12" | 32 @ 8" | 11 @ 6"<br>12 @ 10"<br>11 @ 6" | 21.1 | 4760 |
| 22 | 24'-6" | 3'-3" | 1'-6" | 1'-8" | 4 @ 12"<br>12 @ 15"<br>4 @ 12" | 6 @ 8"<br>18 @ 10"<br>6 @ 8" | 12 @ 6"<br>13 @ 10"<br>12 @ 6" | 23.3 | 5140 |
| 24 | 26'-6" | 3'-4" | 1'-7" | 1'-9" | 4 @ 12"<br>14 @ 15"<br>4 @ 12" | 6 @ 8"<br>21 @ 10"<br>6 @ 8" | 14 @ 6"<br>13 @ 10"<br>14 @ 6" | 25.7 | 6230 |
| 26 | 28'-6" | 3'-5" | 1'-8" | 1'-10" | 4 @ 12"<br>15 @ 15"<br>4 @ 12" | 6 @ 8"<br>23 @ 10"<br>6 @ 8" | 15 @ 6"<br>15 @ 10"<br>15 @ 6" | 28.0 | 6670 |

# STEEL BENDING TABLE

| [illegible] | Bars E d | Bars E e | Bars F a | Bars F b | Bars F c | Bars F d | Bars F e | Bars G a | Bars G b | Bars G c | Bars G d | Bars G e | Bars M h | Bars N1 h | Bars N2 h |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| [illegible] | 2'-2" | 4'-1 3/8" | 1'-0" | 1'-9" | 12'-6" | 1'-1" | 2'-0 3/4" | 2'-9" | 1'-9" | 9'-0" | 1'-1" | 2'-0 3/4" | 2'-7" | 1'-6" | 1'-7" |
| [illegible] | 2'-4" | 4'-7 5/8" | 1'-0" | 2'-0" | 14'-0" | 1'-3" | 2'-4 1/4" | 3'-0" | 2'-0" | 10'-0" | 1'-3" | 2'-4 1/4" | 2'-9" | 1'-8" | 1'-9" |
| [illegible] | 2'-5" | 5'-1 1/4" | 1'-0" | 2'-3" | 15'-6" | 1'-4" | 2'-7 3/8" | 3'-3" | 2'-3" | 11'-0" | 1'-4" | 2'-7 3/8" | 2'-10" | 1'-9" | 1'-10" |
| [illegible] | 2'-6" | 5'-7 1/8" | 1'-0" | 2'-6" | 17'-0" | 1'-5" | 2'-10 1/2" | 3'-6" | 2'-6" | 12'-0" | 1'-5" | 2'-10 1/2" | 2'-11" | 1'-10" | 1'-11" |
| [illegible] | 2'-7" | 6'-0 7/8" | 1'-0" | 2'-9" | 18'-6" | 1'-6" | 3'-1 5/8" | 3'-9" | 2'-9" | 13'-0" | 1'-6" | 3'-1 5/8" | 3'-0" | 1'-11" | 2'-0" |
| [illegible] | 2'-8" | 6'-6 3/4" | 1'-0" | 3'-0" | 20'-0" | 1'-7" | 3'-4 3/4" | 4'-0" | 3'-0" | 14'-0" | 1'-7" | 3'-4 3/4" | 3'-1" | 2'-0" | 2'-1" |



Diagrams

| Clear Span Feet | Bars A - 1/2" □ Slab Reinf. No. | Bars A - 1/2" □ Slab Reinf. Length | B[illegible] | Bars N - 1/2" □ Stirrup - Int. Beam No. | Bars N - 1/2" □ Stirrup - Int. Beam Length |
|---|---|---|---|---|---|
| 16 | 15 | 24'-0" | [illegible] | 38 | 5'-2" |
| 18 | 17 | 24'-0" | [illegible] | 41 | 5'-6" |
| 20 | 19 | 24'-0" | [illegible] | 43 | 5'-8" |
| 22 | 20 | 24'-0" | [illegible] | 46 | 5'-10" |
| 24 | 22 | 24'-0" | [illegible] | 50 | 6'-0" |
| 26 | 24 | 24'-0" | [illegible] | 54 | 6'-2" |

## General Notes & Design Data

B.P.R. 1918 Specifications; typical 15-Ton truck loading; 30% impact allowance, paving or ballast not to exceed 80 lb. per sq.ft.; steel in tension - 16000 lb. per sq.ft. concrete in compression - 650 lb. per sq.in.; reinf. concrete in shear - 110 lb. per sq.in.

Concrete to be 1:2:4 mixture; maximum size of aggregate - 1½". All reinforcing steel to be deformed bars (square twisted bars not to be considered as deformed). Where splicing of reinforcement is necessary, bars are to be lapped 40 diameters.

Railings to suit local conditions to be placed on curbs where necessary, and provision made therefore when pouring curb.

Curbs, slab and beam-stems must be poured at the same time so as to allow no initial set to take place between them.

## CONCRETE DECK GIRDER SPANS
## 16'-0" to 26'-0" Spans 20'-0" Roadway
## U.S. Bureau of Public Roads

S. Bureau of Public Roads.

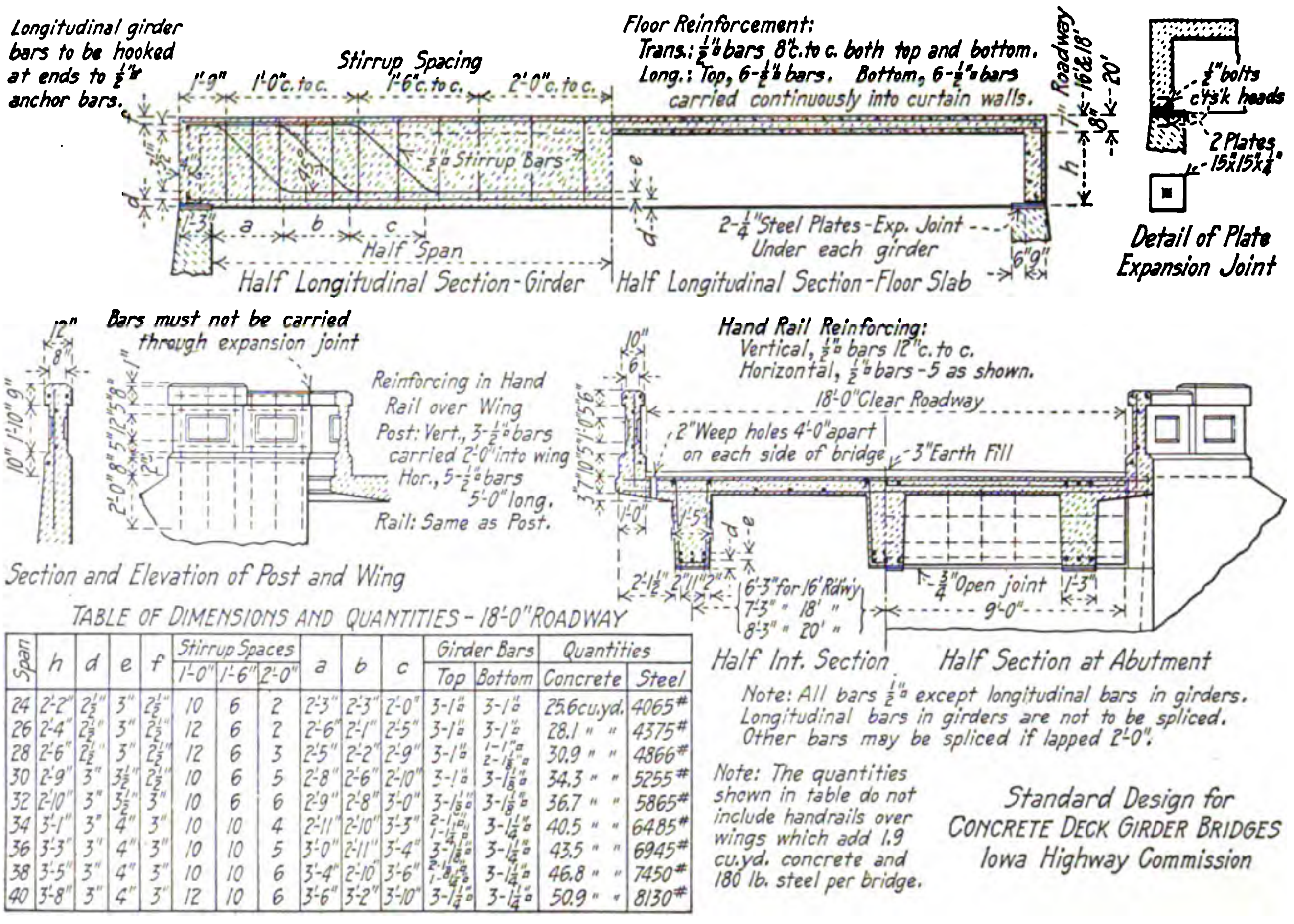



TABLE OF DIMENSIONS AND QUANTITIES – 18'-0" ROADWAY

| Span | h | d | e | f | Stirrup Spaces 1'-0" | Stirrup Spaces 1'-6" | Stirrup Spaces 2'-0" | a | b | c | Girder Bars Top | Girder Bars Bottom | Quantities Concrete | Quantities Steel |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 24 | 2'-2" | 2 1/2" | 3" | 2 1/2" | 10 | 6 | 2 | 2'-3" | 2'-3" | 2'-0" | 3-1"□ | 3-1"□ | 25.6 cu. yd. | 4065# |
| 26 | 2'-4" | 2 1/2" | 3" | 2 1/2" | 12 | 6 | 2 | 2'-6" | 2'-1" | 2'-5" | 3-1"□ | 3-1"□ | 28.1 " " | 4375# |
| 28 | 2'-6" | 2 1/2" | 3" | 2 1/2" | 12 | 6 | 3 | 2'-5" | 2'-2" | 2'-9" | 3-1"□ | 1-1"□ 2-1 1/8"□ | 30.9 " " | 4866# |
| 30 | 2'-9" | 3" | 3 1/2" | 2 1/2" | 10 | 6 | 5 | 2'-8" | 2'-6" | 2'-10" | 3-1"□ | 3-1 1/8"□ | 34.3 " " | 5255# |
| 32 | 2'-10" | 3" | 3 1/2" | 3" | 10 | 6 | 6 | 2'-9" | 2'-8" | 3'-0" | 3-1 1/8"□ | 3-1 1/8"□ | 36.7 " " | 5865# |
| 34 | 3'-1" | 3" | 4" | 3" | 10 | 10 | 4 | 2'-11" | 2'-10" | 3'-3" | 2-1 1/8"□ 1-1 1/4"□ | 3-1 1/4"□ | 40.5 " " | 6485# |
| 36 | 3'-3" | 3" | 4" | 3" | 10 | 10 | 5 | 3'-0" | 2'-11" | 3'-4" | 3-1 1/8"□ | 3-1 1/4"□ | 43.5 " " | 6945# |
| 38 | 3'-5" | 3" | 4" | 3" | 10 | 10 | 6 | 3'-4" | 2'-10" | 3'-6" | 2-1 1/8"□ 1-1 1/4"□ | 3-1 1/4"□ | 46.8 " " | 7450# |
| 40 | 3'-8" | 3" | 4" | 3" | 12 | 10 | 6 | 3'-6" | 3'-2" | 3'-10" | 3-1 1/4"□ | 3-1 1/4"□ | 50.9 " " | 8130# |

FIG. 8. REINFORCED CONCRETE DECK BRIDGE.

The T-beam bridge contains less material than the slab bridge and is especially adapted to spans of from 20 ft. to 35 ft. The T-beam bridge has the added advantage that the roadway can be widened. It has the disadvantage that the cost of construction is more than for a slab bridge.

For the design of a reinforced concrete T-beam bridge, see Fig. 22.

**DECK GIRDERS.**—Details of standard reinforced concrete deck girder bridges as designed by the Iowa Highway Commission are given in Fig. 8. Data and quantities for bridges with spans of 24 ft. to 40 ft. and an 18-ft. roadway are given in the cut. The bridge is anchored to the abutment at one end and slides on steel plates at the other end.

The relative quantities of concrete and reinforcing steel in deck and through girder bridges as designed by the Iowa Highway Commission are given in Table V.

### TABLE V.
COMPARATIVE QUANTITIES FOR THROUGH-GIRDER AND DECK-GIRDER BRIDGES.
IOWA HIGHWAY COMMISSION.
Roadway 18 ft.

| Span, Feet. | Cu. Yd. Concrete. | | Lb. Reinf. Steel. | |
|---|---|---|---|---|
| | Through. | Deck. | Through. | Deck. |
| 24 | 33.5 | 25.6 | 4,520 | 4,065 |
| 26 | 37.5 | 28.1 | 5,024 | 4,375 |
| 28 | 41.8 | 30.9 | 5,568 | 4,886 |
| 30 | 46.5 | 34.3 | 6,198 | 5,255 |
| 32 | 51.5 | 36.7 | 6,697 | 5,865 |
| 34 | 56.9 | 40.5 | 7,372 | 6,485 |
| 36 | 62.6 | 43.5 | 7,972 | 6,945 |
| 38 | 68.3 | 46.8 | 8,570 | 7,450 |
| 40 | 74.3 | 50.9 | 9,416 | 8,130 |

### TABLE VI.
DATA AND QUANTITIES FOR THROUGH-DECK GIRDER BRIDGES. WISCONSIN HIGHWAY COMMISSION.
Designed for a 15-ton road roller.

| Span, Ft. | Roadway, Ft. | Depth of Slab, In. | Depth of Beam Below Slab, In. | Width of Beam, In. | Concrete, Cu. Yd. | Reinforcing, Lb. |
|---|---|---|---|---|---|---|
| 20 | 18 | 8 | 15 | 15 | 21.2 | 3,580 |
| 30 | 18 | 8 | 21 | 15 | 31.8 | 5,690 |
| 40 | 18 | 8 | 24 | 18 | 44.0 | 9,480 |
| 45 | 18 | 8 | 27 | 18 | 54.0 | 12,460 |
| 20 | 20 | 8½ | 21 | 18 | 22.0 | 3,720 |
| 30 | 20 | 8½ | 21 | 18 | 35.3 | 6,250 |
| 40 | 20 | 8½ | 27 | 18 | 48.0 | 10,100 |
| 45 | 20 | 8½ | 30 | 18 | 60.5 | 12,460 |
| 20 | 24 | 8 | 21 | 18 | 25.0 | 4,580 |
| 30 | 24 | 8 | 21 | 18 | 39.0 | 7,590 |
| 40 | 24 | 8 | 24 | 18 | 55.0 | 12,660 |
| 45 | 24 | 8 | 27 | 18 | 67.0 | 15,860 |

The economy of the deck girder type is not as great as indicated by the relative quantities of concrete and steel for the reason that the form work is more complicated for the deck than for the through girder bridges.

Details of a reinforced concrete combined deck and through girder bridge with a span of 35 ft. and a 20-ft. roadway, as designed by the Wisconsin Highway Commission are given in Fig. 9. Data and quantities for through-deck girder bridges as designed by the Wisconsin Highway

## TABLE VII.

### Data and Quantities for Through Girder Bridges. Wisconsin Highway Commission.

| Span, Ft. | Roadway, Ft. | Depth of Slab, In. | Depth of Girder, In. | Width of Girder, In. | Concrete, Cu. Yd. | Reinforcing, Lb. |
|---|---|---|---|---|---|---|
| 25 | 16 | 13 | 54 | 15 | 30.5 | 4,590 |
| 35 | 16 | 13 | 66 | 21 | 52.5 | 7,780 |
| 40 | 16 | 13 | 72 | 24 | 67.0 | 9,700 |
| 25 | 16 | 14 | — | — | 34.0 | 5,210 |
| 30 | 18 | 14 | 61 | 18 | 45.0 | 6,860 |
| 35 | 18 | 14 | 69 | 21 | 59.0 | 9,160 |
| 40 | 18 | 14 | — | — | 67.0 | 9,700 |

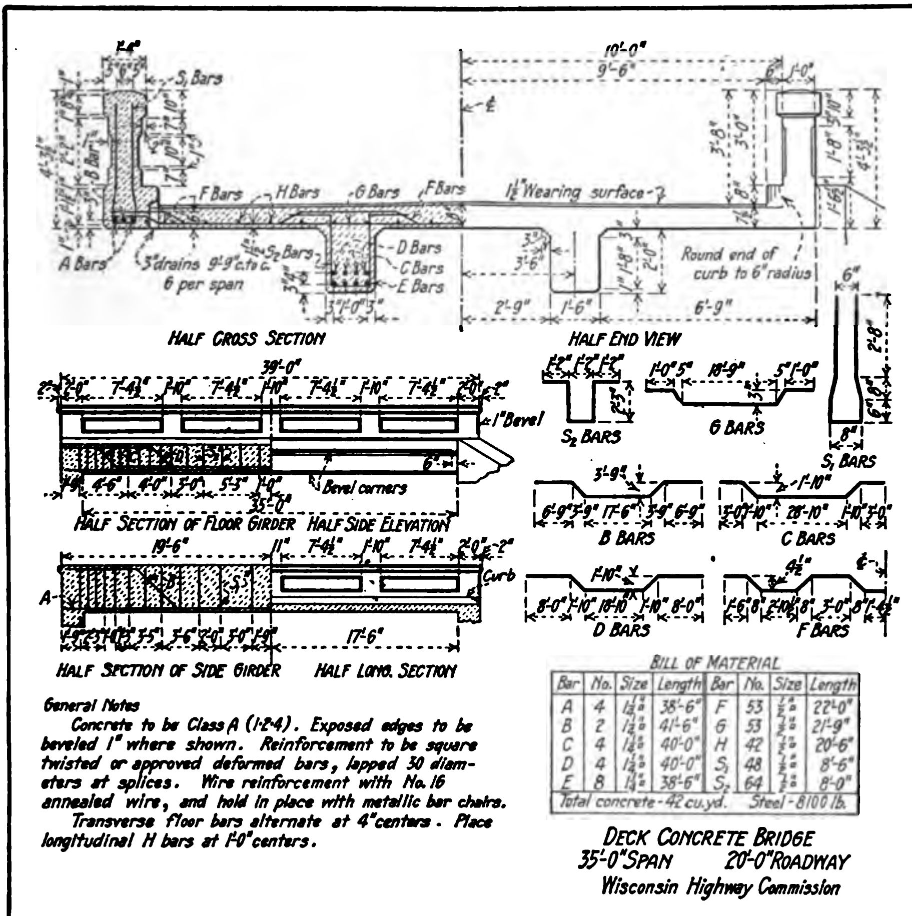



Fig. 9. Reinforced Concrete Through-deck Girder Bridge. Wisconsin Highway Commission.

Commission are given in Table VI. This type of bridge is very economical for a wide roadway. A comparison of the quantities in Table VI and Table VII shows that the through-deck type of girder is more economical than the through girder bridge.

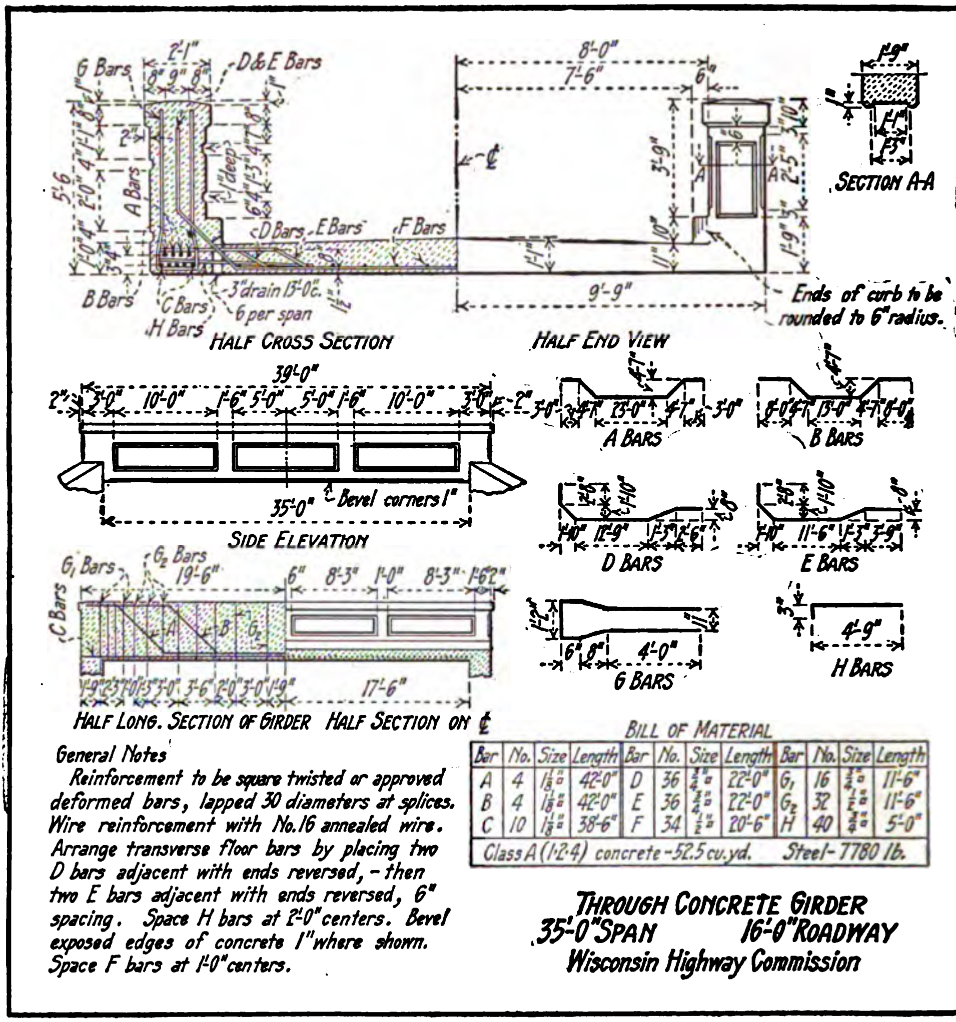



FIG. 10. REINFORCED CONCRETE THROUGH GIRDER BRIDGE. WISCONSIN HIGHWAY COMMISSION.

**THROUGH GIRDER BRIDGES.**—Details of a reinforced concrete through girder bridge, as designed by the Wisconsin Highway Commission are given in Fig. 10.

Details of a reinforced concrete through girder bridge as designed by the Michigan State Highway Department are given in Fig. 11.

The surface of the concrete on the roadway is covered with a bituminous coating as described in the cut.

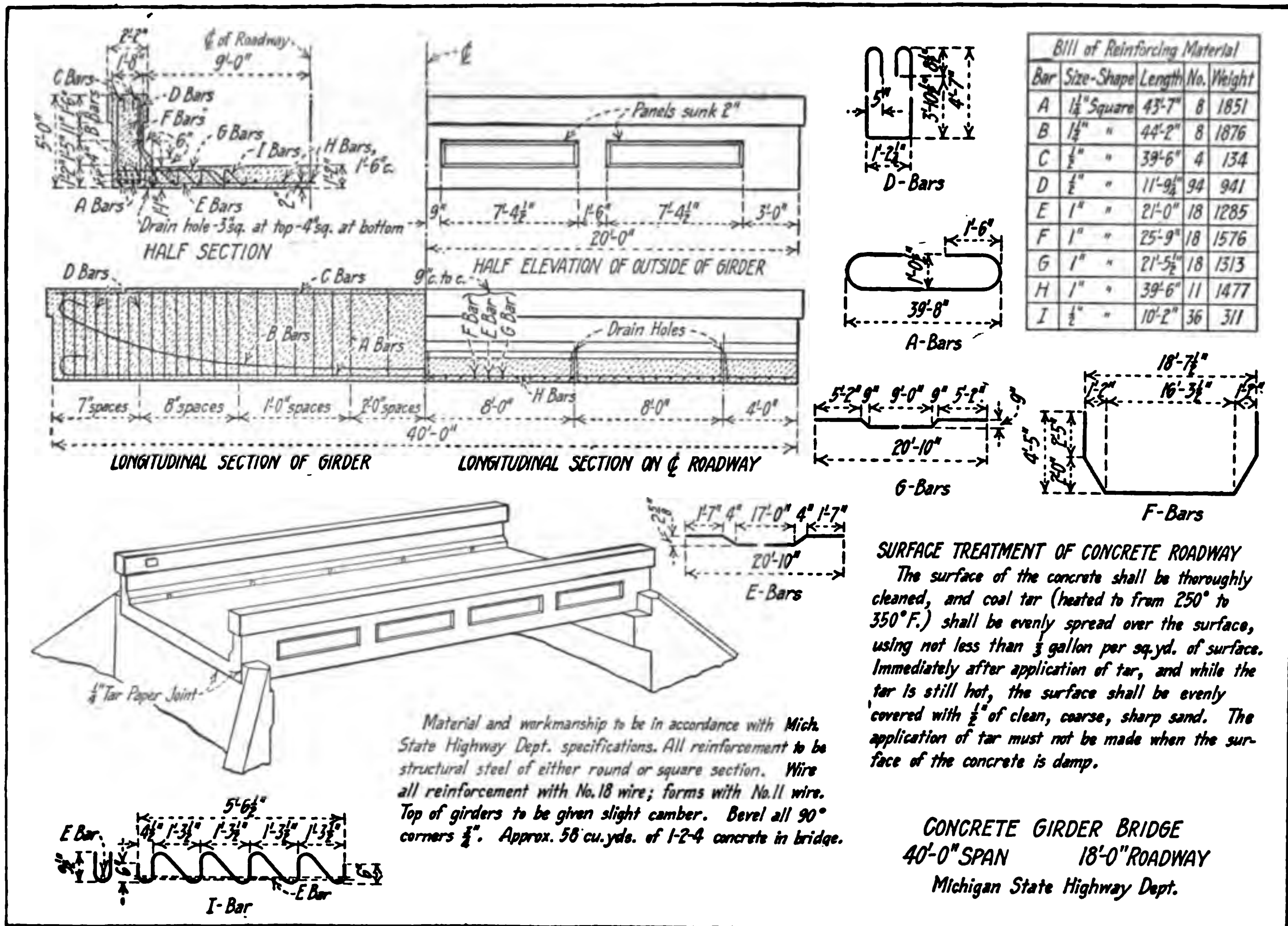
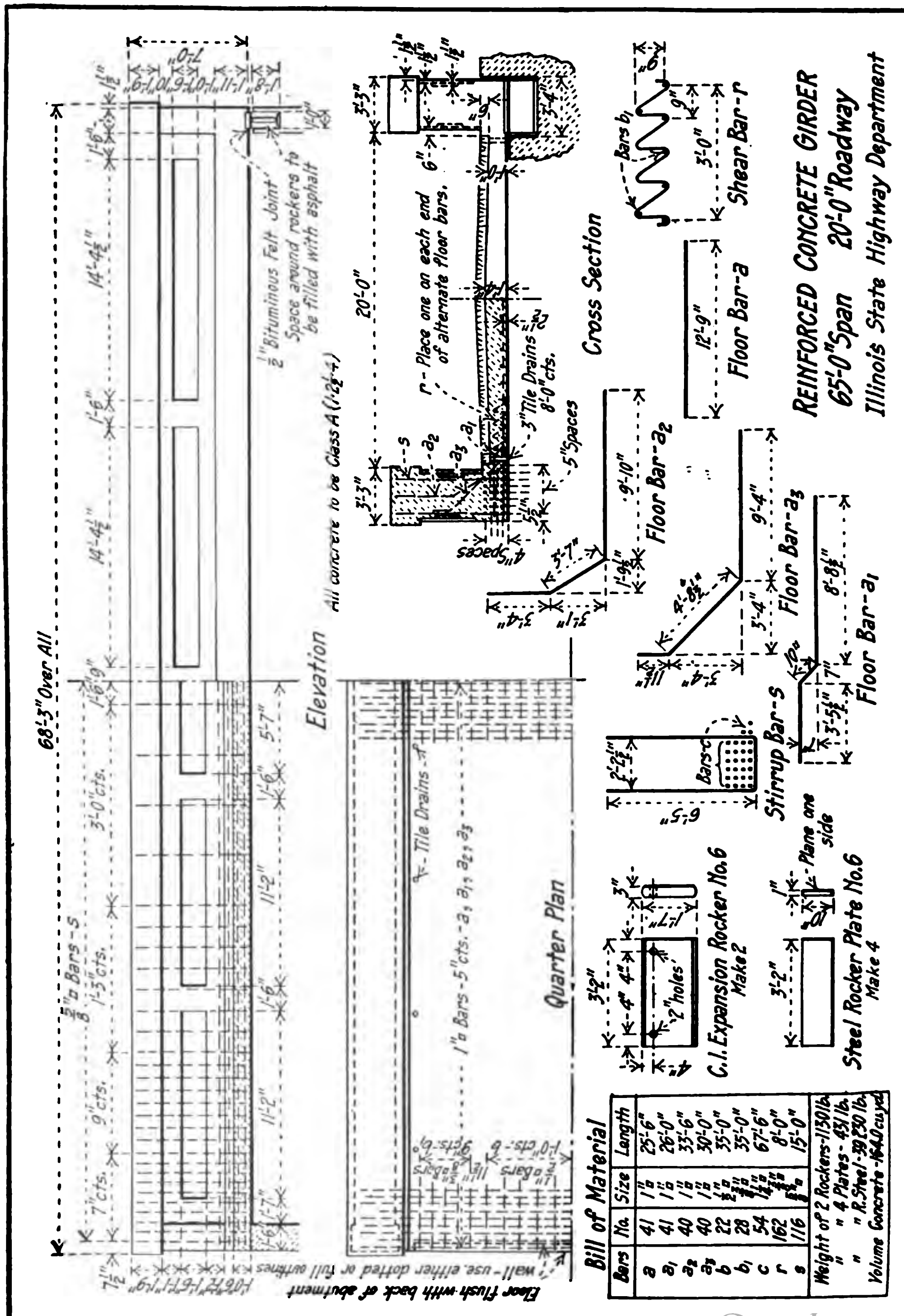



FIG. 11. REINFORCED CONCRETE THROUGH GIRDER BRIDGE. MICHIGAN STATE HIGHWAY DEPARTMENT.




| Bars | No. | Size | Length |
|---|---|---|---|
| a | 41 | 1" □ | 25'-6" |
| a1 | 41 | 1" □ | 26'-0" |
| a2 | 40 | 1" □ | 33'-6" |
| a3 | 40 | 1" □ | 30'-0" |
| b | 22 | 5/8" □ | 35'-0" |
| b1 | 28 | 3/4" □ | 35'-0" |
| c | 54 | 1¼" □ | 67'-6" |
| r | 162 | 3/8" □ | 8'-0" |
| s | 116 | 5/8" □ | 15'-0" |

Weight of 2 Rockers - 1130 lb.
" " 4 Plates - 431 lb.
" " R. Steel - 39230 lb.
Volume Concrete - 164.0 cu. yd.

FIG. 12. REINFORCED CONCRETE THROUGH GIRDER BRIDGE. ILLINOIS HIGHWAY COMMISSION.

Details of a reinforced concrete through girder bridge with a span of 65 ft. and a 20 ft. roadway designed by the Illinois Highway Commission are given in Fig. 12. Data and quantities for through girder spans varying from 30 ft. to 65 ft., and for 16-ft., 18-ft. and 20-ft. roadways are given in Table VIII.

## TABLE VIII.

### Data and Quantities for Through Girder Bridges.
### Illinois Highway Commission.

| Span, Ft. | Roadway, Ft. | Depth of Slab at Center, In. | Depth of Girder, In. | Width of Girder, In. | Concrete, Cu. Yd. | Reinforcing, Lb. |
|---|---|---|---|---|---|---|
| 30 | 16 | 13 | 58 | 16 | 34.6 | 7,270 |
| 30 | 18 | 14½ | 59 | 16 | 39.6 | 8,560 |
| 30 | 20 | 16 | 60 | 16 | 45.1 | 9,990 |
| 35 | 16 | 13 | 58 | 16 | 39.8 | 9,180 |
| 35 | 18 | 14½ | 60 | 16 | 45.9 | 10,640 |
| 35 | 20 | 16 | 67 | 16 | 53.9 | 12,050 |
| 40 | 16 | 13 | 60 | 17 | 47.1 | 11,130 |
| 40 | 18 | 14½ | 66 | 17 | 55.4 | 12,940 |
| 40 | 20 | 16 | 60 | 21 | 65.4 | 15,820 |
| 45 | 16 | 13 | 59 | 21 | 57.8 | 14,430 |
| 45 | 18 | 14½ | 66 | 21 | 68.3 | 16,210 |
| 45 | 20 | 16 | 69 | 22 | 78.9 | 18,720 |
| 50 | 16 | 13 | 66 | 21 | 67.9 | 16,910 |
| 50 | 18 | 14½ | 72 | 21 | 78.5 | 18,980 |
| 50 | 20 | 16 | 76 | 22 | 91.3 | 21,850 |
| 55 | 16 | 13 | 72 | 21 | 77.7 | 19,740 |
| 55 | 18 | 14½ | 72 | 26 | 96.5 | 24,060 |
| 55 | 20 | 16 | 78 | 26 | 110.4 | 26,810 |
| 60 | 16 | 13 | 74 | 26 | 97.6 | 24,340 |
| 60 | 18 | 14½ | 80 | 26 | 111.5 | 27,520 |
| 60 | 20 | 16 | 79 | 31 | 133.5 | 33,100 |
| 65 | 16 | 13 | 79 | 31 | 123.8 | 29,810 |
| 65 | 18 | 14½ | 79 | 36 | 147.3 | 35,790 |
| 65 | 20 | 16 | 84 | 36 | 164.0 | 39,230 |

2 cast rockers weigh 328 lb. for 30-ft. span and 1,130 lb. for 65-ft. span. 4 steel plates weigh 163 lb. for a 30-ft. span and 431 lb. for 65-ft. span.

In the test of a reinforced concrete through girder bridge with 40-ft. span and 18-ft. roadway, made by the Illinois Highway Commission, Proceedings American Society for Testing Materials, Vol. XIII, pp 884– 922, the stress measurements indicated that points of contraflexure occurred at points 15 in. from the side of the girder, the girder slab acting as a simple beam 15.5 ft. long. The Iowa Highway Commission assumes that the point of contraflexure is 1 ft. inside the girders. On the basis of economy the through girder type of bridge is limited to a roadway of about 20 ft.

For details of the design of a through reinforced concrete girder bridge see Fig. 25.

The expansion ends of all through girder bridges designed by the Illinois Highway Commission rest on cast iron rockers that bear on steel plates. The following discussion of expansion of girder bridges is taken from the Fourth report of the Commission.

"Two methods of providing for expansion in girder bridges have been used and both have proved satisfactory. In one method, the wing walls of one abutment are entirely separated from the abutment wall proper, the latter being free to move at the top with the expansion or contraction of the superstructure. The wing walls are designed to be self-supporting. As girder spans designed by the commission have so far been limited to 60 feet, the amount of movement either way from the normal is small and is taken up by deflection of the main wall or a slight

rocking of the wall on the footing. Earth pressure against the wall is of little importance in this connection as it but tends to reduce the tension in the girder steel during expansion and to cause the abutment wall to follow the superstructure during contraction. It does not increase the stress in the compression area of the girder as the load is applied at the bottom of the girder tending by this eccentricity of application to reverse the dead and live load stresses in the girder.

"This method has been found to be entirely successful, but is somewhat objectionable as a slight movement of the wings due to earth pressure and unequal settlement sometimes causes the wing walls to move forward slightly at the top, making a somewhat unsightly off-set between the wing and abutment walls. This has never been more than 2 or 3 inches for the highest walls, but as it is not understood by the ordinary observer, an impression of weakness is sometimes caused.

"The present method of providing for expansion is to design the abutments and wings in the ordinary way, separating the superstructure completely from one of the abutments by a thick paper joint and supporting each girder at the free end on a single cast iron rocker of large diameter. The reaction is transmitted to the girder and abutment from the rocker through planed structural steel plates stiffened with I beams when necessary. The rocker surfaces in contact with the bearing plates are turned to insure perfect bearing on the plates. The diameter of the rocker is made proportional to the load imposed per lineal inch, in the same manner as is commonly used in proportioning roller nests for steel bridges. The upper and lower plates are bedded in the concrete of the superstructure and abutment. The rocker is located in a pocket built in the abutment. This pocket is filled with a soft asphalt to prevent the entrance of water or dirt and to protect the metal from corrosion.

"The rocker method of providing for expansion has proved very satisfactory, and is but little more expensive than the other method, especially when it is considered that the wings may be tied to the main wall when rockers are used, and advantage taken of the mutual support thus obtained." Fig. 12 shows how the rockers are arranged at the free end of the girders.

**EXPANSION ROCKERS.**—The Illinois Highway Commission requires that all reinforced concrete through or deck girder bridges designed as unconstrained structures shall be provided with cast-iron expansion rockers at one end of each span. The rockers to have a thickness not less than 2½ in. for spans of 45 ft. and less, and not less than 3 in. for spans over 45 ft., but in no case shall the unit compressive stress exceed $9{,}000 - 40l/r$ lb. per sq. in. Rockers to have bearing surfaces turned to uniform radius and smooth surface, and have 2-in. holes to facilitate handling. Bearing plates to be steel plates not less than 1 in. thick with planed bearing surfaces. Rocker pockets are made 2 in. longer than rockers. The top of the rocker is placed ½ in. above surface of concrete. The rockers are blocked up by means of sticks with cross-section of one inch, and the pocket is filled with asphalt. The top plate is then blocked up with sticks during the pouring of the girder. The space between girder and abutment is filled with bituminous felt.

The Minnesota Highway Commission has the same specifications for expansion rockers.

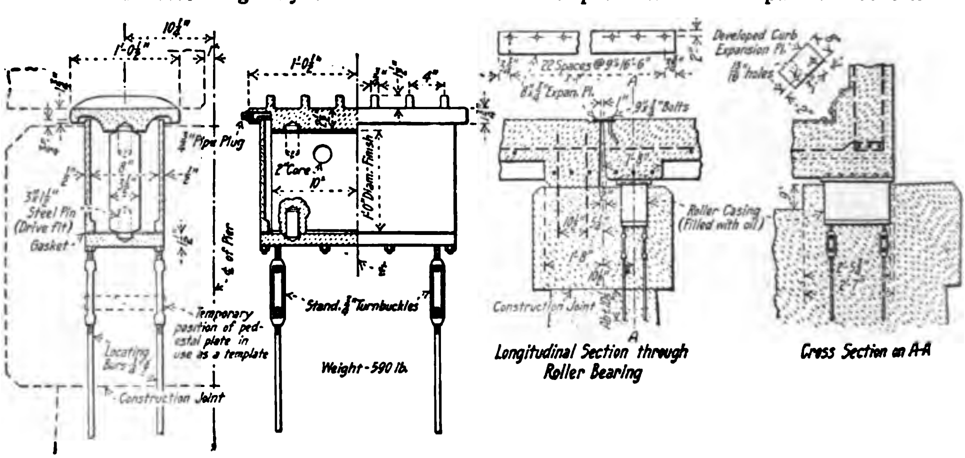
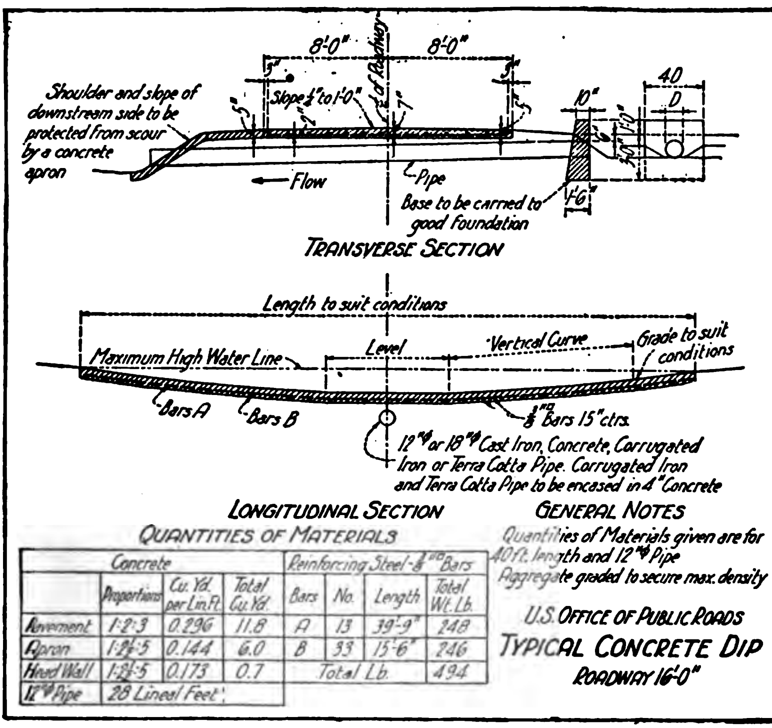



Fig. 12a. Segmental Bearings for Concrete Girders. Michigan State Highway Department.

*Segmental Roller Bearings for Concrete Girders.*—The segmental roller bearings for concrete girders shown in Fig. 12a were designed by Mr. C. V. Dewart for the Michigan State Highway Department. The bearing consists of a hollow casting containing in a medium of heavy oil the segmental rollers which fit into the girder shoe. The pier is first poured up to the under side of the coping, and in this run it is necessary to place the locating bars. In placing these bars the pedestal plate may be used as a template. The turnbuckles are then placed on top of the locating bars and the pedestal castings on top of the turnbuckles, and leveled up by means of the turnbuckles. The vertical castings are held in place by means of temporary wooden struts. The open joint between the ends of the girders is protected with an expansion plate.

**OVERFLOW BRIDGES.**—In many localities in the west and southwest, streams which at flood carry a large volume of water are entirely dry or have a very small flow except for the occasional flood flow. In many cases the building of a permanent bridge with a waterway sufficiently large to carry the flood flow is not possible on account of the cost. The overflow bridge must be so designed as to carry the normal flow, and at the same time withstand the action of the heavy current, and not lodge drift; and that when the flood has subsided will leave the roadway free from flood deposit. Two types of overflow bridges are used.




| | Concrete | | | Reinforcing Steel - ⅜" ϕ Bars | | | |
|---|---|---|---|---|---|---|---|
| | Proportions | Cu. Yd. per Lin. Ft. | Total Cu. Yd. | Bars | No. | Length | Total Wt. Lb. |
| Pavement | 1:2:3 | 0.296 | 11.8 | A | 13 | 39'-9" | 248 |
| Apron | 1:2½:5 | 0.144 | 6.0 | B | 33 | 15'-6" | 246 |
| Head Wall | 1:2½:5 | 0.173 | 0.7 | Total Lb. | | | 494 |
| 12"ϕ Pipe | 28 Lineal Feet | | | | | | |

FIG. 13. STANDARD CONCRETE DIP. U. S. BUREAU OF PUBLIC ROADS.

**1. Concrete Dip.**—Where there is no flow or a small flow, except at flood, the roadway is depressed to make a channel for flood flow, the embankment is paved with concrete on each side to prevent erosion, and a small flow culvert is placed at the lowest point of the channel. The grade of the road descending into the channel and ascending should not exceed 5 per cent. Details of a standard "Concrete Dip" as designed by the U. S. Bureau of Public Roads are given in Fig. 13.

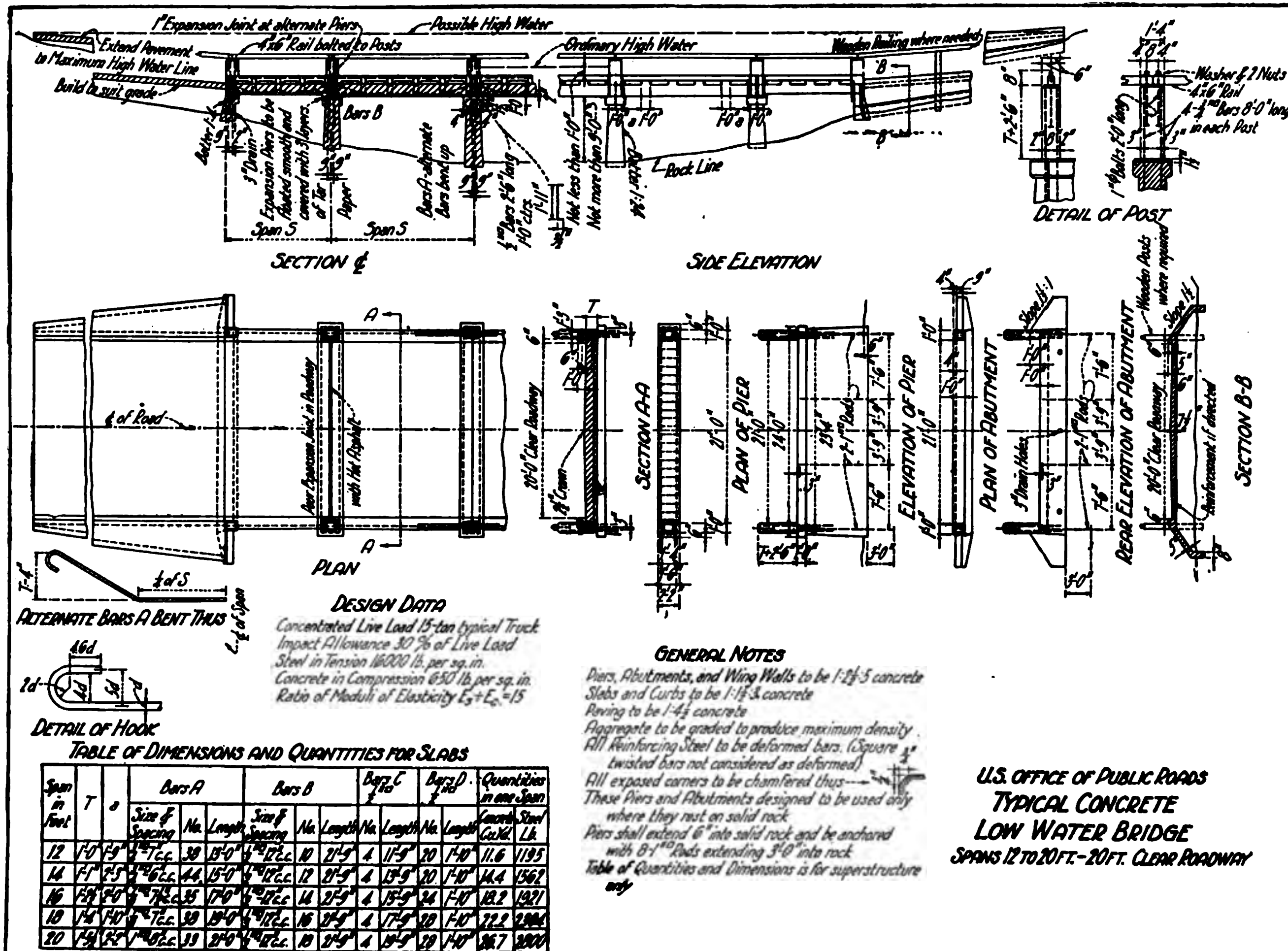



TABLE OF DIMENSIONS AND QUANTITIES FOR SLABS

| Span in Feet | T | a | Bars A Size & Spacing | Bars A No. | Bars A Length | Bars B Size & Spacing | Bars B No. | Bars B Length | Bars C No. | Bars C Length | Bars D No. | Bars D Length | Quantities in one Span Concrete Cu. Yd. | Quantities in one Span Steel Lb. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 12 | 1'-0" | 1'-9" | ¾" - 7" c.c. | 38 | 13'-0" | ½" - 12" c.c. | 10 | 21'-5" | 4 | 11'-9" | 20 | 1'-10" | 11.6 | 1195 |
| 14 | 1'-1" | 2'-5" | ¾" - 6" c.c. | 44 | 15'-0" | ½" - 12" c.c. | 12 | 21'-5" | 4 | 13'-9" | 20 | 1'-10" | 14.4 | 1562 |
| 16 | 1'-2½" | 2'-0" | ⅞" - 7½" c.c. | 35 | 17'-0" | ½" - 12" c.c. | 14 | 21'-5" | 4 | 15'-9" | 24 | 1'-10" | 18.2 | 1921 |
| 18 | 1'-4" | 1'-10" | 1" - 7" c.c. | 38 | 19'-0" | ½" - 12" c.c. | 16 | 21'-5" | 4 | 17'-9" | 28 | 1'-10" | 22.2 | [illegible] |
| 20 | 1'-5½" | 2'-2" | 1" - 6" c.c. | 33 | 21'-0" | ½" - 12" c.c. | 18 | 21'-5" | 4 | 19'-9" | 28 | 1'-10" | 26.7 | [illegible] |

FIG. 14. STANDARD CONCRETE OVERFLOW HIGHWAY BRIDGE. U. S. BUREAU OF PUBLIC ROADS.

2. **Overflow Bridge.**—Where it is desirable to take care of normal storms the waterway is provided by means of several culverts, and the remainder of the bridge consists of fill between reinforced concrete walls. The railing should be of a type that will not hold the drift and will pass the flood flow freely. Details of a standard reinforced concrete overflow bridge as designed by the U. S. Bureau of Public Roads, are given in Fig. 14. The rail on top of the curb is made so that it will wash away in flood, and will not collect drift. The openings in the curb will produce an increased velocity of the water over the top of the bridge floor that will keep the surface clear of silt.

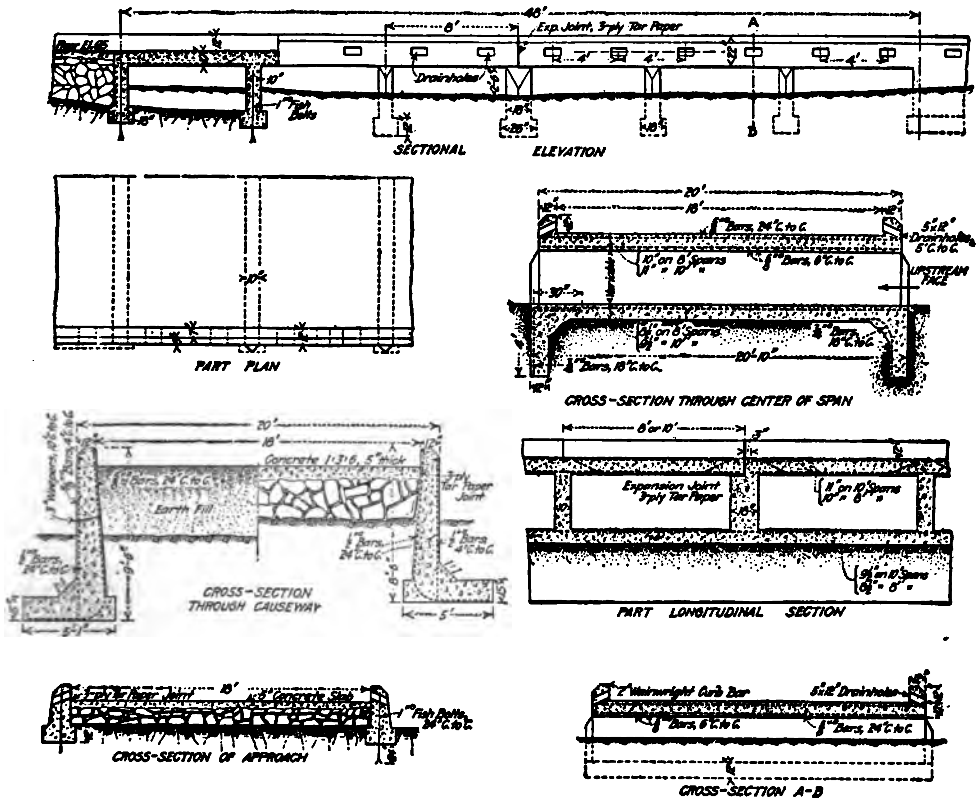



FIG. 15. CONCRETE OVERFLOW BRIDGE, BEXAR COUNTY, TEXAS.

The low water bridge shown in Fig. 15 was built in Bexar County, Texas. The bridge consists of a reinforced concrete slab bridge resting on concrete piers, and a filled portion between concrete retaining walls. The total length of the bridge is 252 ft. In 1915 the cost was $21.70 per lineal foot of bridge. For rock footings the foundations are carried 12 in., into rock and are anchored with fish bolts. The foundations on clay and gravel are made of a continuous slab footing, and at the upstream and down stream edges the wall extends four feet below the footing slab. Fill is placed between the retaining walls, and the roadway is paved with concrete.

The curbing is 12" wide and 12" high, and has openings at intervals. These openings cause a current that cleans the roadway of silt. There is no rail.

**CONTINUOUS GIRDERS.**—A continuous reinforced concrete girder bridge of three spans is shown in Fig. 16. The roadway for a single track electric railway is laid on ballast carried on

the reinforced concrete floor. The structure is reinforced as shown in the cut. There is no expansion joint in the structure. The left hand abutment and the piers rest on solid rock, the right hand abutment rests on clay.

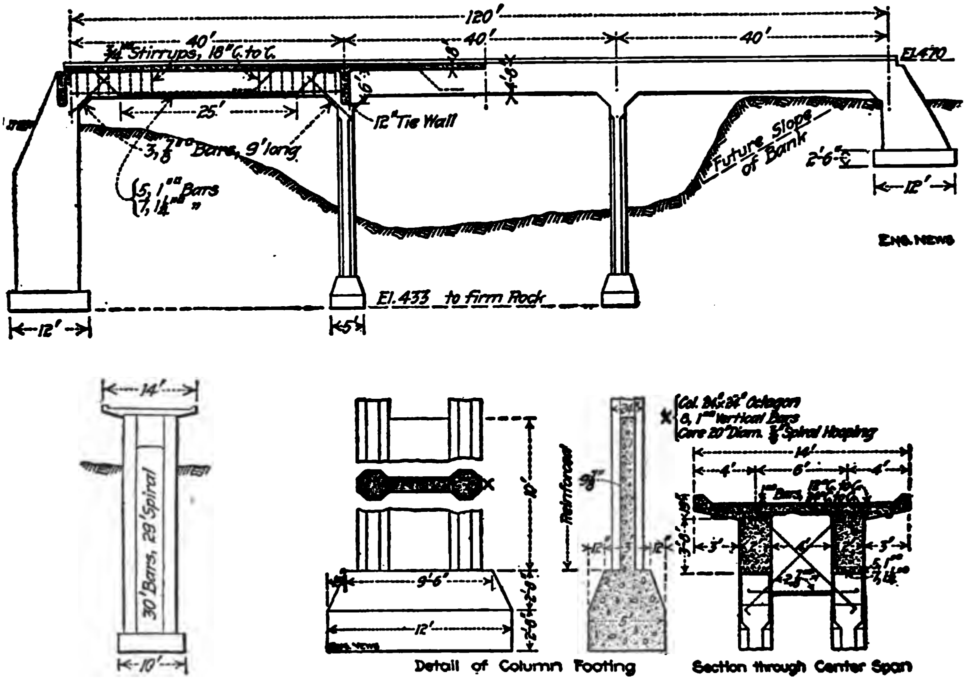



FIG. 16. CONTINUOUS REINFORCED CONCRETE GIRDER BRIDGE.

Continuous reinforced concrete bridges require heavier foundations than for bridges with simple spans, if settlement is to be prevented. The shear near the piers over which the girder is continuous is greater than at the ends of simple girders, and there is no economy in continuous reinforced concrete girders over simple span reinforced concrete girders. Continuous reinforced concrete girders usually require more material, and more careful workmanship, than simple span reinforced concrete girders and are not to be recommended for highway bridges.

**CANTILEVER REINFORCED CONCRETE GIRDERS.**—The Colfax-Larimer reinforced concrete viaduct shown in Fig. 17, was designed as a cantilever bridge. This design was worked out by the designing engineer, Mr. H. S. Crocker, M. Am. Soc. C. E., to obviate the difficulties incident to the construction of a continuous structure and at the same time obtain a more economical structure than could be obtained with simple spans. The spans vary from 40 ft. to 50 ft. The suspended span has a span length equal to one-half the total span length. All columns are supported on independent footings. The main viaduct carries a roadway and electric tracks, each of which is carried on a viaduct with four columns in each tower, making four columns in line in a transverse direction. The roadway floor slab and electric railway floor slab are connected by a slab free to move at the ends parallel to the roadway. A very unusual rain occurred during construction and several columns near the west end settled several inches, without causing any serious cracks or injuring the structure. These columns were easily jacked back into place. The same settlement with a continuous bridge would have wrecked the structure. Details of the suspension link are shown in Fig. 17. The cantilever reinforced concrete bridge is especially suitable for viaducts.

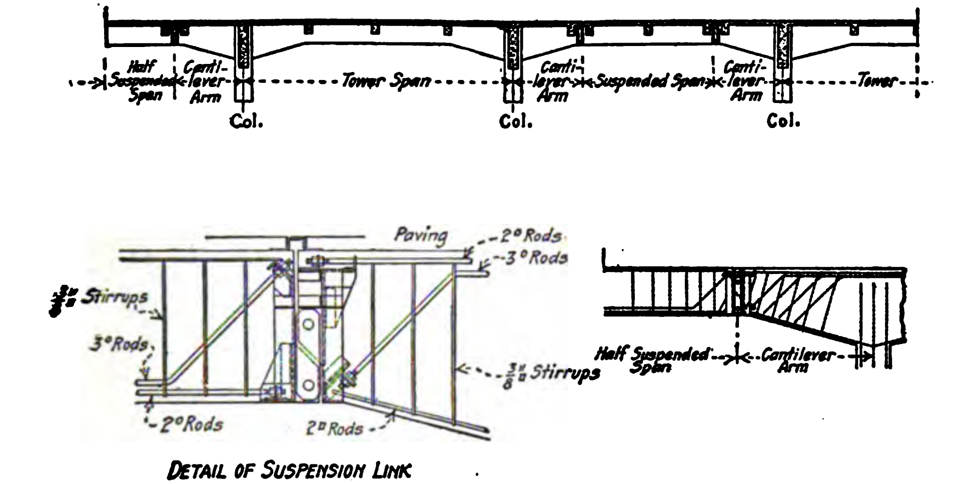



FIG. 17. REINFORCED CONCRETE CANTILEVER BRIDGE. COLFAX-LARIMER VIADUCT, DENVER, COLORADO.

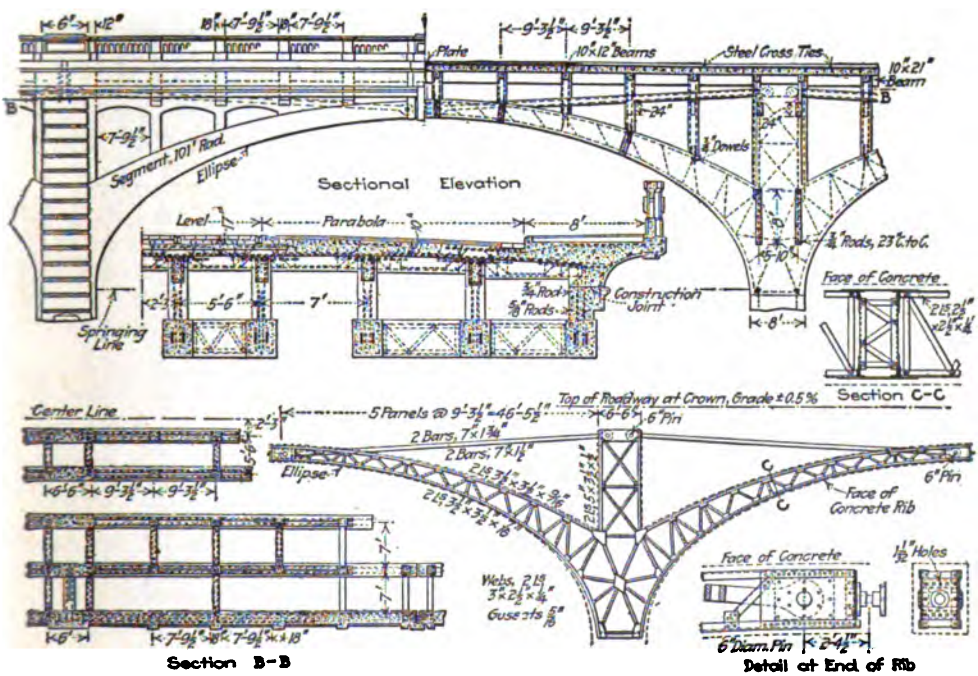



FIG. 17a. CANTILEVER ARCH BRIDGE. HANOVER STREET BRIDGE, BALTIMORE, MD.

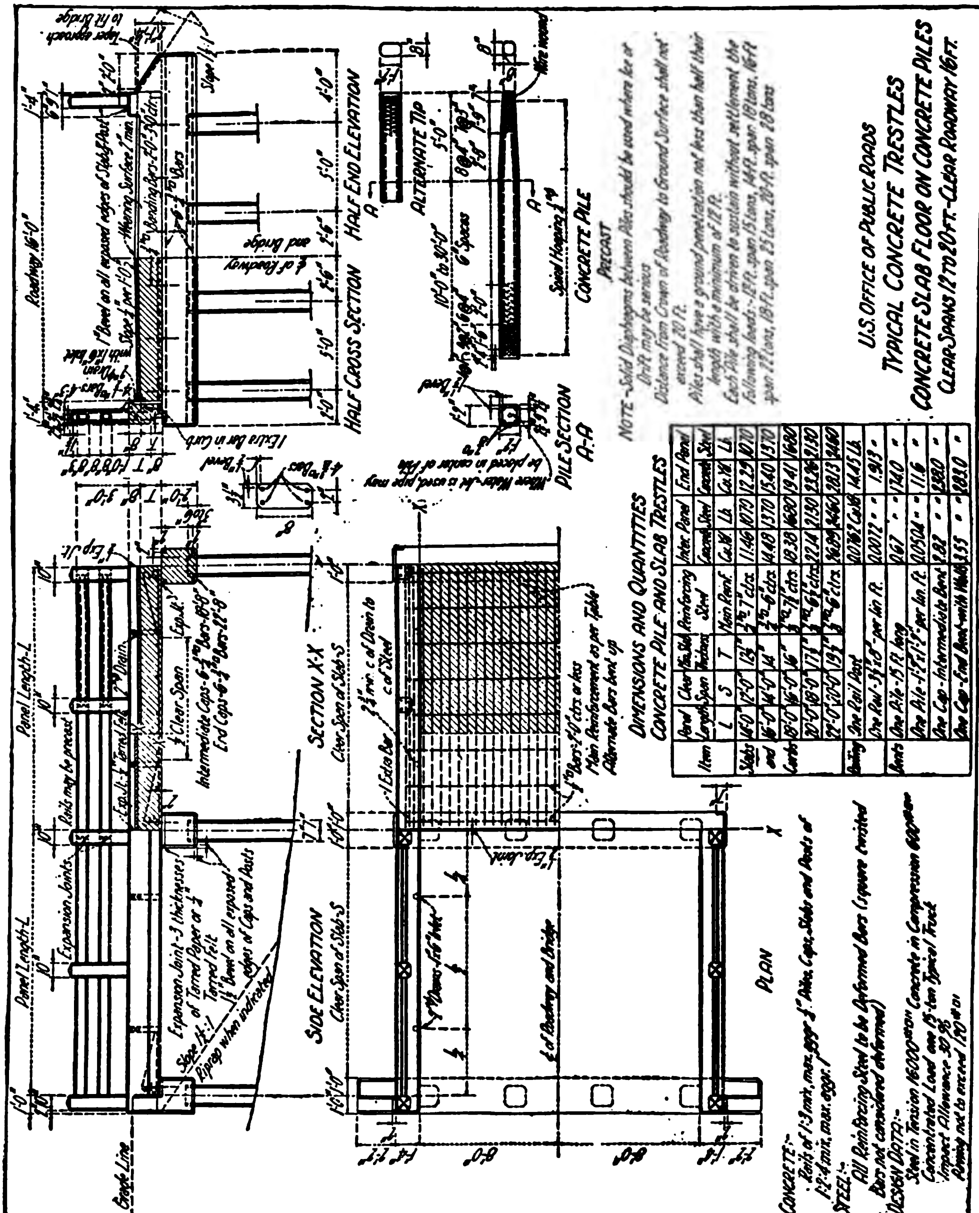



FIG. 18. REINFORCED CONCRETE TRESTLE BRIDGE. U. S. BUREAU OF PUBLIC ROADS.

The cantilever arch bridge shown in Fig. 17a was designed to give the artistic effect of an arch structure and the economy of a cantilever structure. The cantilever reinforcement carries the dead load, while the live load is divided between the steel and the concrete. This bridge is described in Engineering News, March 20, 1917.

**REINFORCED CONCRETE TRESTLE BRIDGES.**—Standard plans for reinforced concrete trestle highway bridges designed by the U. S. Bureau of Public Roads are given in Fig. 18. The bents are composed of four reinforced concrete piles with a reinforced concrete cap. The bents are spaced from 14 ft. to 22 ft. centers, or 12 ft. to 20 ft. in clear. The bridges are designed for a 15-ton truck with axles spaced 10 ft. centers, and wheels with 6 ft. gage. Two-thirds of the load is to be carried on the rear axle. With ballast of 8 in. or less each wheel, is assumed as uniformly distributed over 5 feet square. An allowance of 30 per cent is made for impact. Concrete for slabs, caps and piles is to be 1 : 2 : 4 mix with aggregate not more than 1 in. in diameter. Allowable stress on steel reinforcement is 16,000 lb. per sq. in. Allowable compression in concrete is 600 lb. per sq. in.

Reinforced concrete precast piles are made with an average diameter of not less than 12 in. and the diameter at the point not less than 8 in. The length shall not exceed 30 times the average diameter for piles driven through firm soil, or 15 times the average diameter for piles driven to rock through loose, wet soil or filled ground. When piles are not supported they are designed as columns. Piles shall be loaded as determined by formula or by tests, but not to exceed a maximum of 300 lb. per sq. in. on the smallest section, or 30 tons per pile as a maximum. For arches, continuous spans and high abutments the load shall not exceed 25 tons per pile. Reinforcing steel shall be rigidly fastened in place before casting. Centers of main reinforcing bars shall not be nearer the surface of the concrete than 2½ inches. The concrete shall be placed continuously. Forms shall be straight and true, shall be water-tight, and shall not be removed within 24 hours after the concrete is placed. All exposed surface of piles shall be given a rubbed surface. Piles shall be cured at least 40 days with a temperature of not less than 40° F., or 30 days with a temperature of not less than 60° F. Piles shall be at least 30 days old when driven. When concrete piles are lifted or moved they shall be supported at the quarter points, and they shall be so designed that the unit stress produced by handling shall not exceed 650 lb. per sq. in. in compression or 16,000 lb. per sq. in. in the steel reinforcement.

Piles shall be driven with drop hammers weighing not less than the weight of the pile, or with steam hammers weighing not less than two-thirds the weight of the pile. The tops of piles shall be protected by cushions of wood or rope. The safe load shall be determined by the Engineering News formula where $s$ is the average penetration for the last 5 blows with a drop hammer or last 30 blows with a steam hammer. See specifications in Appendix II.

**DESIGN OF REINFORCED CONCRETE BRIDGES.**—Detail designs are given for (1) a slab bridge, (2) a T-beam bridge, and (3) a through girder bridge. The designs follow the specifications in Appendix II. All references are to formulas in Chapter XVIII.

### Design of a 13-ft. Span Reinforced Concrete Slab-Bridge.

1. **General Description of Bridge.**—This bridge consists of a reinforced concrete floor slab supported at each end on the abutments, with reinforced concrete railing on each side of the roadway. The railing is not considered as assisting in carrying the load. The ends of the slab will be so supported that expansion and contraction will cause only nominal stresses in the slab and no negative moment will be produced. A wearing surface weighing 30 lb. per sq. ft. of roadway will be provided.

2. **LOADS. Dead Load.**—The dead load consists of the weight of the floor slab at 150 lb. per cu. ft. and the wearing surface. The railing will be made self supporting so need not be carried by the slab.

**Live Load.**—This bridge will be designed for Class D1 loading, given in the specifications, which provides for a 20-ton concentrated load on two axles 12 feet apart or a uniform live load of 125 lb. per sq. ft. of roadway.

**Impact.**—An allowance of 30 per cent of the live load stresses will be made for impact.

**Wind Load.**—The wind load need not be considered in this type of structure.

3. **Dimensions.**—Span, 15′ 0″ center to center of end bearings; width of roadway, 20′ 0″; width of railing, 10″; height of railing, 4′ 0″ above top of the slab.

4. **DESIGN OF THE SLAB.**—In the design of the slab one wheel load will be considered as distributed on a line parallel to the abutments and having a length as given the formula

$$e = \tfrac{2}{3}L + c$$

with a maximum of six feet, where

$e$ = effective width of distribution of load, in feet.
$L$ = span in feet.
$c$ = width of wheel in feet.

For $L = 15$, $e = \frac{2}{3} \times 15 + 1.67 = 11.67$ ft. so the maximum of six feet controls.

The rear wheel load per foot of width is $P_r = 14{,}000 \div 6 = 2{,}330$, and adding 30 per cent impact $P_r = 2{,}330 \times 1.30 = 3{,}030$ lb. For the front wheel the load per foot of width is $P_f = 6{,}000 \div 6 = 1{,}000$ lb. and adding 30 per cent for impact $P_f = 1{,}300$ lb.

The maximum live load bending moment will occur when the rear wheel is at mid-span.

The live load moment per foot of width for $P_r$ at the center of the span is

$$M_L = \tfrac{1}{4} \times 3{,}030 \times 15 = 11{,}400 \text{ ft.-lb.}$$

The live load bending moment per foot of width for the uniform load is

$$M_L = \tfrac{1}{8}w \cdot l^2 = \tfrac{1}{8} \times (1.30 \times 125) \times 15^2 = 4{,}570 \text{ ft.-lb.}$$

The first case gives the maximum of $M_L = 11{,}400$ ft.-lb. The maximum live load shear occurs at the right end when $P_r$ is just over the right support, and is

$$V_L = \frac{(P_r + P_f) \times 11.4}{15} = \frac{4{,}330 \times 11.4}{15} = 3{,}290 \text{ lb.}$$

The dead load moment per ft. of width assuming a 15 in. slab, weight 188 lb. per sq. ft., and considering the wearing surface is

$$M_D = \tfrac{1}{8} \times 218 \times 15^2 = 6{,}140 \text{ ft.-lb.}$$

The dead load shear per ft. of width assuming a 15 in. slab is

$$V_D = \frac{218 \times 15}{2} = 1{,}640 \text{ lb.}$$

The total bending moment is

$$M = 11{,}400 + 6{,}140 = 17{,}540 \text{ ft.-lb.} = 211{,}000 \text{ in.-lb.}$$

The total shear is

$$V = 3{,}290 + 1{,}640 = 4{,}930 \text{ lb.}$$

For the unit stresses given in the specifications, the required depth to the center of the steel is

$$d = 0.0965\sqrt{\frac{M}{b}} = 0.0965\sqrt{\frac{211{,}000}{12}} = 12.75 \text{ in.} \tag{6c}$$

Adding $1\frac{3}{4}$ in. below the center of the steel, the total thickness is $12.75 + 1.75 = 14.50$ in. A total thickness of $14\frac{1}{2}$ in. will be used, making $d = 12.75$, provided this thickness is satisfactory for shear. The area of steel per foot of width required to develop this slab is (From Fig. 2, Chap. XVIII, for $f_s = 16{,}000$ lb. and $f_c = 650$ lb., $p = 0.0077$.)

$$A = 0.0077 \cdot b \cdot d = 0.0077 \times 12 \times 12.75 = 1.18 \text{ sq. in.}$$

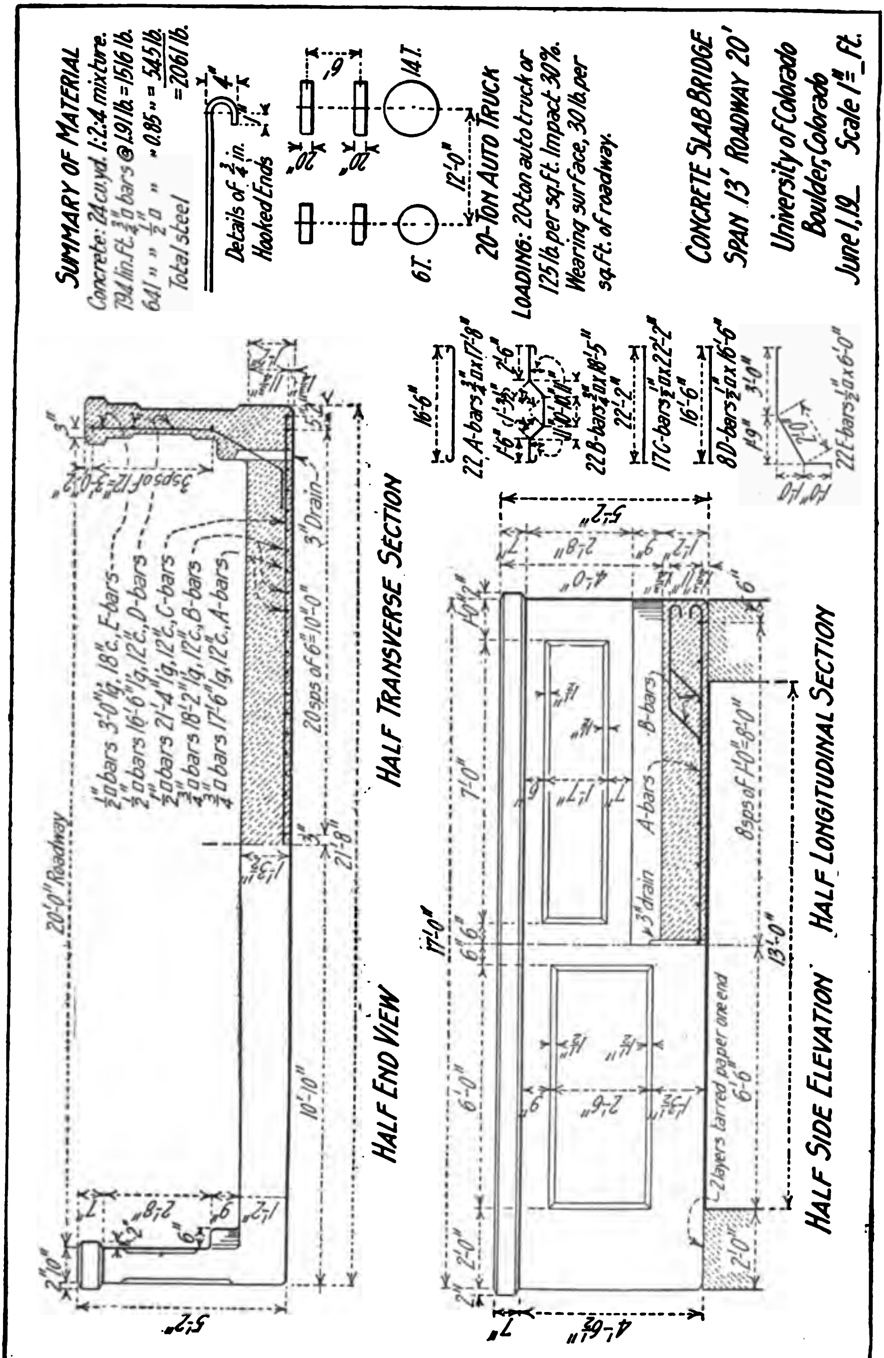



FIG. 19. REINFORCED CONCRETE SLAB BRIDGE. DESIGNED FOR KETCHUM'S SPECIFICATIONS, $D_1$ LOADING.

25

for the unit stresses given in the specifications. Bars ¾-in. square spaced 6 in. c. to c. provide an area of 1.13 sq. in. per ft. width, and will be used for the reinforcement perpendicular to the abutments, and ½-in. square bars 12 in. c. to c. will be used parallel to the abutments. (The area of bars ¾-in. square spaced 6 in. c. to c. is slightly under but these bars will be used.)
The maximum unit shear is

$$f_v = \frac{V}{b \cdot j \cdot d} = 1.15 \frac{V}{b \cdot d} = 1.15 \times \frac{4,930}{12 \times 12.75} = 37 \text{ lb. per sq. in.}$$

requiring no shear reinforcement, but some will be provided to increase the reliability of the structure.

The maximum unit bond stress considering one-fourth of the bars as bent up is (67)

$$f_u = \frac{V}{\Sigma o \cdot j \cdot d} = 1.15 \frac{V}{\Sigma o \cdot d} = 1.15 \times \frac{4,930}{4.5 \times 12.75} = 99 \text{ lb. per sq. in.}$$

One-half of the bars will be bent up but half of these bars will be effective for bond as may be seen from Fig. 19.

5. **Detail Drawings.**—The detail drawings for this bridge are shown in Fig. 19.

### DESIGN OF A 28-FT. CONCRETE T-BEAM SPAN.

1. **General Description of Bridge.**—This bridge is to have a clear span of 28′ 0″. It will consist of a reinforced concrete slab supported on several reinforced concrete beams running between the abutments. A wearing surface weighing 30 lb. per sq. ft. will be provided.

2. **LOAD. Dead Load.**—The dead load consists of the weight of the structure considering reinforced concrete to weigh 150 lb. per cu. ft.; and the wearing surface weighing 30 lb. per sq. ft.

**Live Load.**—This bridge will be designed for Class $D_2$ loading, given in the specifications, which provides for a 15-ton auto truck with axles spaced 10 ft. c. to c. and wheels spaced 6 ft. c. to c., the width of rear tires being 15 in.; or a uniform live load of 100 lb. per sq. ft. of roadway.

**Impact.**—An allowance of 30 per cent of the live load will be made for impact on the slabs and beams.

3. **Dimensions.**—Span, clear 28′ 0″, c. to c. supports about 30′ 0″. Width of roadway, 16′ 0″, c. to c. outside beams about 17′ 0″.

Three intermediate and two outside beams will be used, making the spacing about 4′ 3″.

4. **DESIGN OF SLAB.**—The slab will be reinforced on top and bottom and will be assumed as continuous. The rear wheels of the auto truck will determine the section.
The effective width of the slab for moment is

$$e = \tfrac{2}{3}(l + c) = \tfrac{2}{3}(4.25 + 1.25) = 3.00 \text{ ft.}$$

where $e$ = effective width of distribution of load in feet.
$l$ = span in feet = 4.25 ft.
$c$ = width of tire = 15 in. = 1.25 ft.
The wheel load per foot width of slab is

$$P = 1.30 \times 10,000 \div 3.00 = 4,330 \text{ lb.}$$

Since the slab is to be reinforced on top and bottom it may be considered as continuous and the bending moment taken as ⅔ the moment for a simply supported slab. The live load moment per foot of width is $M_L = \tfrac{2}{3}(\tfrac{1}{2} \times 4,330 \times \tfrac{1}{2} \times 3 - \tfrac{1}{2} \times 4,330 \times \tfrac{1}{2} \times 0.625) = 1,710$ ft.-lb. Assume a 5½ in. slab and a wearing surface of 30 lb. per sq. ft. The dead load moment per foot of width is

$$M_D = \tfrac{2}{3}(\tfrac{1}{8} w \cdot l^2) = \tfrac{1}{12}(69 + 30)3^2 = 74 \text{ ft.-lb.}$$

The total bending moment at the center and over the beams is

$$M = 1{,}710 + 74 = 1{,}784 \text{ ft.-lb.} = 21{,}408 \text{ in.-lb.}$$

per foot of width.

For the unit stresses given in the specifications the required depth to the center of the steel is

$$d = \sqrt{\frac{M}{R \cdot b}} = \sqrt{\frac{21{,}408}{107.5 \times 12}} = 4.06 \text{ in.} \tag{6b}$$

A slab with a total thickness of $5\frac{1}{4}$ in. will be used placing the steel on top and bottom 1 in. from the surface, making $d = 4.25$ in., providing this slab will be sufficient for shear. The area of steel per foot width required to develop this slab is

$$A = 0.0077\, b \cdot d = 0.0077 \times 12 \times 4.25 = 0.393 \text{ sq. in.}$$

for the unit stresses given in the specifications. Bars $\frac{1}{2}$ in. square spaced $7\frac{1}{2}$ in. c. to c. provide an area of 0.40 sq. in. and will be used if satisfactory for bond stress. Bars $\frac{3}{8}$ in. square spaced about 12 in. c. to c. will be placed near the top and bottom of the slab and running parallel to the beams. This reinforcement is to assist in distributing the loads and to provide for temperature changes.

The effective width for shear is taken equal to that for moment with a minimum of 3 ft. Since the effective width is 3 ft. the unit shear will all be punching shear, and will have a value of

$$f_v = \frac{V}{b \cdot d} = \frac{\frac{1}{2} \times 4{,}330 + 1.5 \times 99}{4.25 \times 12} = 45 \text{ lb. per sq. in.}$$

The allowable value is 120 lb. per sq. in. Since all of the shear is considered as punching shear bond stress need not be calculated.

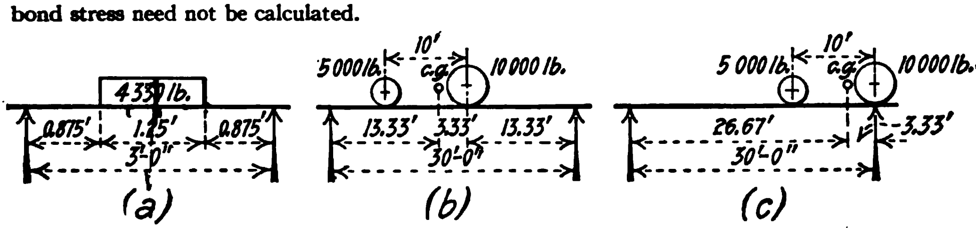



FIG. 20.

4. **Design of Intermediate Beams.**—The bending moment in one beam due to live load of 100 lb. per sq. ft. plus 30 per cent impact is

$$M_L = \tfrac{1}{8} w \cdot l^2 = \tfrac{1}{8}(4.25 \times 130)30^2 = 62{,}000 \text{ ft.-lb.}$$

The end shear in one beam due to the live load of 100 lb. per sq. ft. plus 30 per cent impact is

$$V_L = \tfrac{1}{2} w \cdot l = \tfrac{1}{2}(4.25 \times 130)30 = 8{,}300 \text{ lb.}$$

The proportion of the front and rear wheels of the auto truck carried by one joist is $4.25 \div 6 = 0.708$. The position of the wheels of the 15-ton auto truck for maximum moment is shown in (*b*), Fig. 20, and for maximum shear in (*c*), Fig. 20. The bending moment in one beam due to the auto truck and 30 per cent impact is

$$M_L = 1.30 \times 0.708 \frac{15{,}000 \times 13.33^2}{30} = 82{,}000 \text{ ft.-lb.}$$

The shear in one beam due to the auto truck and 30 per cent impact is

$$V_L = 1.30 \times 0.708 \frac{15{,}000 \times 26.67}{30} = 12{,}270 \text{ lb.}$$

The auto truck gives larger values for both shear and moment than those given by the uniform load, so these values will be used.

The dead load shear and moment cannot be determined until the weight of the beams is known. The stem of the beams will be assumed to be 16 in. wide and 24 in. deep, and will weigh $16 \times 26 \times 150 \div 144 = 430$ lb. per ft. The dead load carried by one beam is

| | | |
|---|---|---|
| Wearing surface, | $4.25 \times 30$ | = 130 lb. per ft. |
| Slab, | $4.25 \times 66$ | = 280 " " " |
| Beam, | | = 430 " " " |
| Total | | = 840 lb. per ft. |

The dead load bending moment is

$$M_D = \tfrac{1}{8} w \cdot l^2 = \tfrac{1}{8} \times 840 \times 30^2 = 94{,}500 \text{ ft.-lb.}$$

The dead load shear is

$$V_D = \tfrac{1}{2} w \cdot l = \tfrac{1}{2} \times 840 \times 30 = 12{,}600 \text{ lb.}$$

The total bending moment is

$$M = M_L + M_D = 82{,}000 + 94{,}500 = 176{,}500 \text{ ft.-lb.} = 2{,}120{,}000 \text{ in.-lb.}$$

The total shear is

$$V = V_L + V_D = 12{,}270 + 12{,}600 = 24{,}870 \text{ lb.}$$

The slab acts as the flange of a T-beam. The width on each side which may be considered as effective is $4 \times 5.25 = 21$ in., making a total width of $42 + 16 = 58$ in. assuming stem to be 16 in. wide. This value is greater than the distance center to center of beams, so the distance center to center of beams = 51 in. will be used, providing it is not greater than one-fourth of the span, or $28 \div 4 = 7$ ft. = 84 in. The minimum section allowed if fully reinforced for shear is

$$b'd = \frac{V}{j \cdot f_v} = \frac{24{,}870}{0.90 \times 120} = 230 \text{ sq. in.}$$

assuming $j = 0.90$. Using $b' = 16$ in., $d$ would equal 14.4 in. The depth should not be much less than $\frac{1}{12}$ of the span = 30 in. The most economical depth is given by the formula

$$d = \sqrt{\frac{r \cdot M}{f_s b'}} + \frac{t}{2}$$

where $r$ = ratio of unit cost of steel in place to unit cost of concrete in place, using same units for steel and concrete. A value of $r = 70$ will be used

$$d = \sqrt{\frac{70 \times 2{,}120{,}000}{16{,}000 \times 16}} + \frac{5.25}{2} = 26.7 \text{ in.}$$

Try $d = 28$ in.

$$R = \frac{M}{b \cdot d^2} = \frac{2{,}120{,}000}{51 \times 28^2} = 53$$

which will probably be satisfactory.

$$s = \frac{t}{d} = \frac{5.25}{28} = 0.188$$

$$j = \frac{3(1-s) + (s)^2}{3\left(1 - \frac{s}{2}\right) - \frac{f_s}{4n \cdot R}(s)^2} \tag{24}$$

$$= \frac{3(1 - 0.188) + (0.188)^2}{3(1 - .094) - \frac{16{,}000 \times 0.188^2}{4 \times 15 \times 53}} = 0.92$$

The area of steel required is

$$A = \frac{M}{f_s \cdot j \cdot d} = \frac{2{,}120{,}000}{16{,}000 \times 0.92 \times 28} = 5.15 \text{ sq. in.}$$

Ten ¾-in. square bars provide an area of 5.63 sq. in. If ten bars were used six bars could be bent up for web reinforcement. Twelve bars will allow eight to be bent up, which may be found necessary. Eight ⅝-in. square bars and four ¾-in. square bars provide an area of 5.30 sq. in. Twelve bars with eight of them bent up will be used, for six bent-up bars give too great a spacing, the maximum allowable being $\frac{3}{4}d = \frac{3}{4} \times 28 = 21$ in.

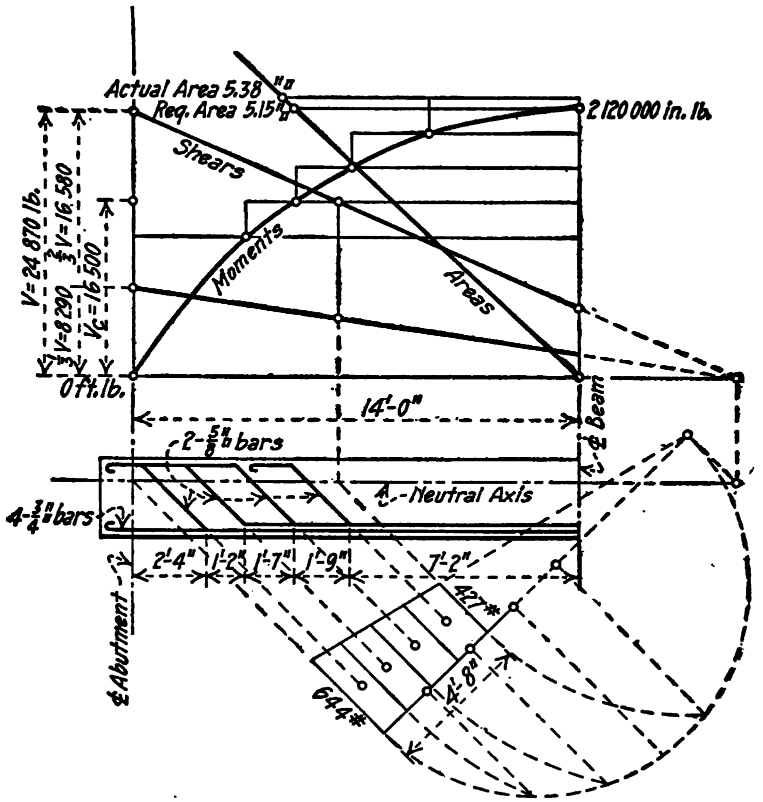



FIG. 21.

The unit stress in the concrete must be determined to see that it does not exceed the allowable value of 650 lb. per sq. in. This requires that $p$ and $k$ be found and substituted in the formula

$$f_c = \frac{f_s \cdot k}{n(1 - k)} \tag{3}$$

$p = A \div b.d = 5.38 \div 51 \times 28 = 0.0038$ if twelve bars are used.

$$k = \frac{pn + \frac{1}{2}\left(\frac{t}{d}\right)^2}{p \cdot n + \frac{t}{d}} = \frac{0.0038 \times 15 + 0.0177}{0.0038 \times 15 + 0.188} = 0.305 \tag{14}$$

or

$$f_c = \frac{16,000 \times 0.305}{15(1 - 0.305)} = 470 \text{ lb. per sq. in.}$$

The assumed section will be adopted. No revision in weight is necessary.

Part of the tensile steel bars will be bent up to provide web reinforcement. The point at which web reinforcement must start is determined from the shear diagram given in Fig. 21. The points at which bars may be bent up are determined from the moment diagram in Fig. 21. The stresses in the bent up bars are determined from the diagonal tension diagram in Fig. 21. The construction of these diagrams will be explained.

The shear and moment diagrams were constructed by calculating the total shear and moment at several points in the beam and plotting their values. The curves were then drawn through these points.

The shearing stress which the concrete may be considered as carrying without web reinforcement is $V_c = f_v \cdot b \cdot j \cdot d = 40 \times 16 \times 0.92 \times 28 = 16{,}500$ lb., the allowable value of $f_v$ being 40 lb. per sq. in. A horizontal line is drawn across the shear diagram, Fig. 21 until it intersects the shear curve. To the right of this intersection no shear reinforcement is needed although it is good practice to provide light vertical stirrups spaced not to exceed $\frac{3}{4}d$. To the left of this intersection the web reinforcement is considered as carrying two-thirds of the shear.

The points at which the bars may be bent up for shear were determined from the moment diagram by plotting the areas provided by each pair of bars, on the moment diagrams to the proper scale. The scale is determined by making the required area equal to the maximum moment. The points at which bars may be discontinued are given by the intersections of the lines representing areas with the moment curve. The unit bond stress considering the eight $\frac{7}{8}$-in. bars to be bent up, depends upon the four $\frac{7}{8}$-in. straight bars, and is

$$f_u = \frac{V}{\Sigma o \cdot j \cdot d} = \frac{24{,}870}{4 \times 3 \times 0.92 \times 29} = 81 \text{ lb. per sq. in.} \tag{69}$$

The allowable bond stress with hooked ends = 120 lb. per sq. in.

5. **Design of Outside Beams.**—The outside beams will be designed as rectangular beams. The top of the beams will project above the top of the slab to provide a curb. The rail will be assumed as concrete weighing 280 lb. per ft. It is evident that the slab adds but little to the strength of the beam for it is located near the neutral axis and comes on one side only.

The specifications require that the same live load be used on the outside beams as for the intermediate beams. Therefore

$$M_L = 82{,}000 \text{ ft.-lb.,} \qquad V_L = 12{,}270 \text{ lb.}$$

The dead load carried byoutside beam is

| | | |
|---|---|---|
| Wearing surface, | $\frac{1}{2} \times 4.25 \times 30 =$ | 64 lb. per ft. |
| Slab, | $1.46 \times 66 =$ | 96 " " " |
| Beam, (assumed) | = | 720 " " " |
| Railing, | = | 280 " " " |
| Total | = | 1,160 lb. per ft. |

The maximum dead load bending moment is

$$M_D = \tfrac{1}{8} \times 1{,}160 \times 30^2 = 130{,}500 \text{ ft.-lb.}$$

The maximum dead load shear is

$$V_D = \tfrac{1}{2} \times 1{,}160 \times 30 = 17{,}400 \text{ lb.}$$

The total bending moment is

$$M = M_L + M_D = 82{,}000 + 130{,}500 = 212{,}500 \text{ ft.-lb.} = 2{,}550{,}000 \text{ in.-lb.}$$

The total shear is

$$V = V_L + V_D = 12{,}270 + 17{,}400 = 29{,}670 \text{ lb.}$$

The width of the beam will be taken as 16 in., the same as the width of stem of the intermediate beams. The minimum depth as determined by the bending moment is

$$d = \sqrt{\frac{M}{R \cdot b}} = \sqrt{\frac{2{,}550{,}000}{107.5 \times 16}} = 38.5 \text{ in.} \tag{6b}$$

A curb projecting 12 in. above the top of the slab will be provided. This requires that the depth be 28 + 12 = 40 in. A depth of 40 in. will be used if satisfactory for shear.

For $b = 16$ in. and $d = 40$ in.

$$R = \frac{M}{b \cdot d^2} = \frac{2{,}550{,}000}{16 \times 40^2} = 99.6$$

From diagram, Fig. 2, Chapter XVIII, $p = 0.0070$ and $j = .880$

$$A = p \cdot b \cdot d = .0070 \times 16 \times 40 = 4.48 \text{ sq. in.}$$

Eight $\frac{3}{4}$-in. square bars were used. $A = 8 \times 0.5625 = 4.50$ sq. in. Maximum unit shearing stress

$$f_v = \frac{V}{b \cdot j \cdot d} = \frac{29{,}670}{16 \times 0.88 \times 40} = 53 \text{ lb. per sq. in.} \tag{68}$$

This exceeds the allowable of 40 lb. per sq. in. It is good practice to bend up some of the bars and to provide some vertical stirrups. The stresses are so small no calculations will be made. Assuming 4 of the 8 bars to be bent up the bond stress at the end is

$$f_u = \frac{V}{\Sigma o \cdot j \cdot d} = \frac{29{,}670}{4 \times 3 \times .88 \times 40} = 70 \text{ lb. per sq. in.} \tag{67}$$

The allowable bond stress with hooked ends = 120 lb. per sq. in.

If four of the horizontal bars are bent up in pairs the bends being spaced $\frac{3}{4}d$ = 30 in. apart the tensile stress in the bent-up bars due to the shear is

$$f_s = \frac{2}{3}\,\frac{0.7\,V \cdot s'}{A \cdot j \cdot d} = \frac{2}{3}\,\frac{0.7 \times 29{,}670 \times 30}{2 \times 0.5625 \times 40} = 9{,}250 \text{ lb. per sq. in.} \tag{74}$$

if no allowance is made for vertical stirrups.

It will usually be found desirable to make the complete graphical solution as explained under intermediate beams.

6. **Detail Drawings.**—Detail drawings are given in Fig. 22.

## Design of a 33-ft. Span Through Concrete Girder Bridge.

1. **General Description of Bridge.**—This bridge is to consist of a reinforced concrete floor slab supported by two reinforced concrete girders at the edges of the roadway. These girders also act as railings. A wearing surface of 30 lb. per sq. ft. of roadway will be provided.

2. **LOADS. Dead Load.**—The dead load for the slab consists of the weight of the slab and wearing surface. The dead load for the girders consists of the weight of the slab, wearing surface and girders. The weight of reinforced concrete is taken as 150 lb. per cu. ft.

**Live Load.**—This bridge will be designed for Class $D_1$ loading which provides for a 20-ton concentrated load or a uniform load of 125 lb. per sq. ft. of roadway for the floor and 125 lb. per sq. ft. for the girders.

**Impact.**—The specifications provide for an allowance for impact of 30 per cent for the floor slab, and of

$$\frac{100}{L + 300} = \frac{100}{335} = 30 \text{ per cent}$$

for the girders.

**Wind Load.**—The wind load need not be considered in this type of structure.

3. **Dimensions.**—Span, 35′ 0″ c. to c. of bearing; width of roadway, 16′ 0″; spacing of girders, about 18′ 0″ c. to c.

4. **DESIGN OF SLAB.**—In designing the slab the rear axle load will be considered as distributed over 12 feet measured parallel to the axle, and one-twelfth of the axle load will be assumed as carried on a width of one foot when calculating moment and one-sixth when calculating shear.

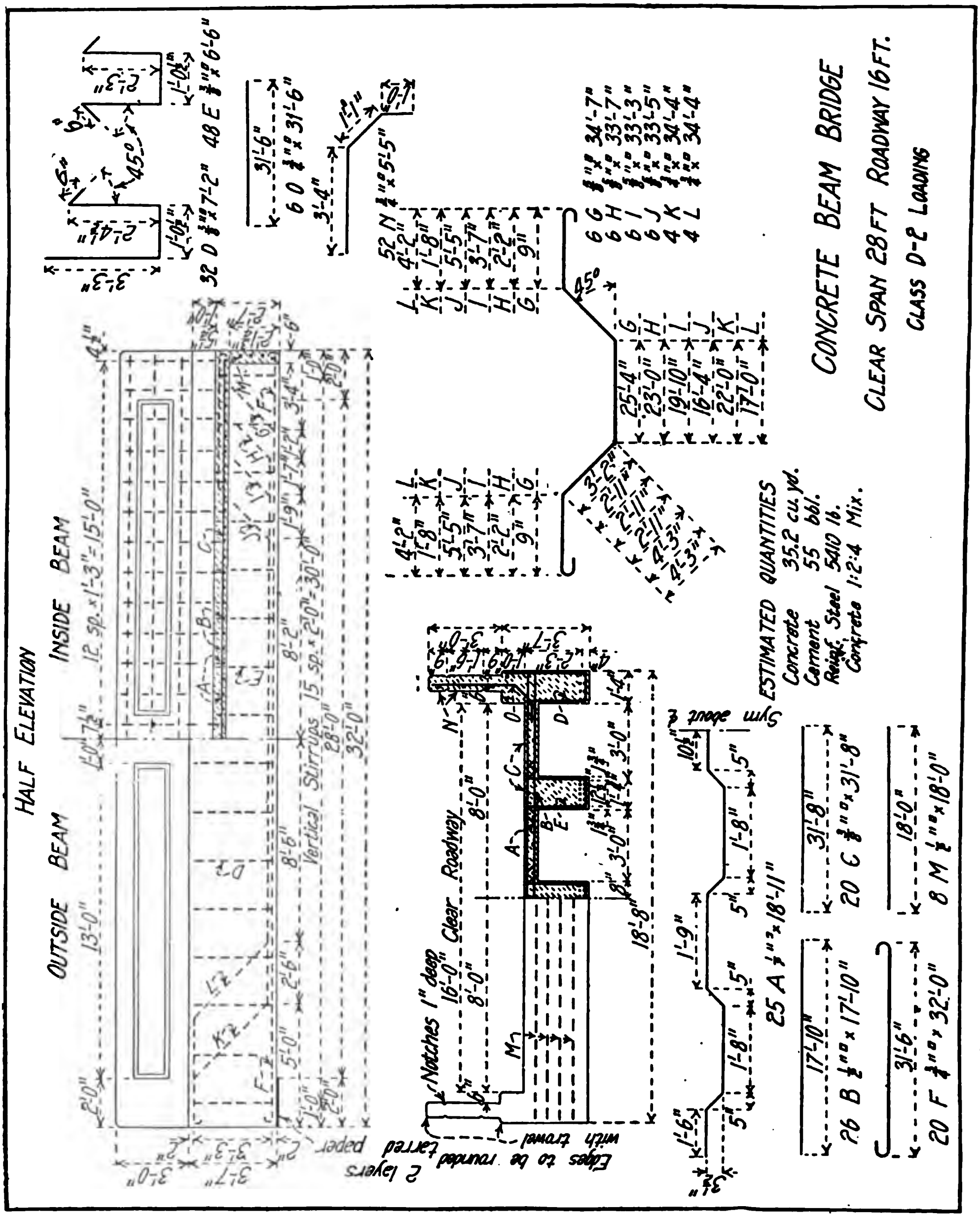



FIG. 22. REINFORCED CONCRETE T-BEAM BRIDGE. DESIGNED FOR KETCHUM'S SPECIFICATIONS, $D_2$ LOADING.

The load per foot width of slab due to the concentrated load, including 30 per cent impact is

$$\frac{1.30 \times 28{,}000}{12} = 3{,}030 \text{ lb.}$$

uniformly distributed over 12 feet and placed in the center of the slab for maximum moment, as shown in (*a*), Fig. 23.

The slab will be considered as simply supported with a span equal to the distance center to center of girders. There will however be some negative moment at the girders which must be provided for, but the assumption of completely fixed ends is not warranted. The maximum live load moment per foot width of slab due to the concentrated load is

$$M_L = \tfrac{1}{2} \times 3{,}030 \times \tfrac{1}{2} \times 18 - \tfrac{1}{2} \times 3{,}030 \times \tfrac{1}{2} \times 6 = 9{,}090 \text{ ft.-lb.}$$

The maximum live load moment due to 125 lb. per sq. ft. of roadway, and including 30 per cent impact is

$$M_L = 1.30[\tfrac{1}{2} \times (125 \times 16) \times \tfrac{1}{2} \times 18 - \tfrac{1}{2} \times (125 \times 16) \times \tfrac{1}{2} \times 8] = 6{,}500 \text{ ft.-lb.}$$

which is less than that due to the concentrated load, so need not be considered.

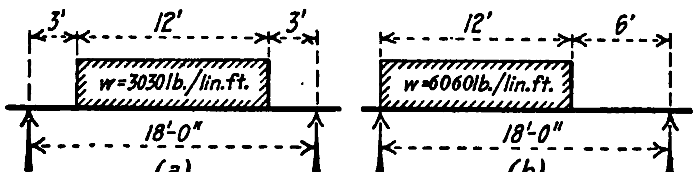



FIG. 23.

The dead load moment per foot width, assuming a 14½ in. slab and considering 30 lb. per sq. ft. for wearing surface is

$$M_D = \tfrac{1}{8} \times 211 \times 18^2 = 8{,}540 \text{ ft.-lb.}$$

The total bending moment per foot width of slab is

$$M = 9{,}090 + 8{,}540 = 17{,}630 \text{ ft.-lb.} = 212{,}000 \text{ in.-lb.}$$

For the unit stresses given in the specifications the required depth to the center of the steel is

$$d = 0.0965\sqrt{\frac{M}{b}} = 0.0965\sqrt{\frac{212{,}000}{12}} = 12.80 \text{ in.} \qquad (6c)$$

Adding 1¾ in. below the center of the steel, the total thickness of slab is 14.55 in. A total thickness of 14½ in. will be used making $d = 12.75$ in., provided this thickness is satisfactory for shear. The area of steel per foot width of slab required to develop this slab is (Fig. 2, Chap. XVIII.)

$$A = 0.0077b\cdot d = 0.0077 \times 12 \times 12.75 = 1.17 \text{ sq. in.}$$

for the unit stresses given in the specifications. Bars ¾ in. square spaced 6 in. c. to c. provide an area of 1.13 sq. in. per ft. width, and will be used for the reinforcement perpendicular to the girders, and ½ in. square bars spaced 12 in. c. to c. parallel to the girders to provide for temperature changes and to distribute the load.

The maximum live load end shear for the concentrated load will occur for the load placed as shown in (*b*), Fig. 23. The load per ft. width of slab for shear is 1.30 × 28,000 ÷ 6 = 6,060 lb. The live load shear is

$$V_L = \frac{6{,}060 \times 12}{18} = 4{,}040 \text{ lb.}$$

The maximum live load shear for the uniform load of 125 lb. per sq. ft. of roadway is

$$V_L = \tfrac{1}{2}(1.30 \times 125) \times 16 = 1{,}300 \text{ lb.}$$

which is less than that due to the concentrated load so need not be considered.
The dead load shear per foot width of slab is

$$V_D = \tfrac{1}{2} \times 211 \times 18 = 1{,}900 \text{ lb.}$$

The total shear per fcot width of slab is

$$V = 4{,}040 + 1{,}900 = 5{,}940 \text{ lb.}$$

The maximum unit shear considering $d = 12$ in. at curb is

$$f_v = \frac{V}{b \cdot j \cdot d} = 1.15 \frac{V}{b \cdot d} = 1.15 \frac{5{,}940}{12 \times 12} = 47 \text{ lb. per sq. in.} \tag{68}$$

The maximum bond considering every third bar as bent up is

$$f_u = \frac{V}{\Sigma o \cdot j \cdot d} = 1.15 \frac{V}{\Sigma o \cdot d} = 1.15 \frac{5{,}940}{6 \times 12} = 95 \text{ lb. per sq. in.} \tag{67}$$

This value for bond stress is calculated on the basis of a simply supported beam and refers to the steel in the bottom of the slab, so applies inside of the point of contra-flexure where the shear is slightly less than considered and where the bent-up bars are at the bottom of the slab. The steel on top and at the end will have a bond stress of $\frac{3}{2} \times 95 = 142$ lb. per sq. in. The rods bent up into the girder will decrease this somewhat.

Shear reinforcement will be provided as shown in the detail in Fig. 24, so the slight excess of the maximum shear over the allowable value will be taken care of. The ends of the tension reinforcement will be hooked to provide additional bond strength.

5. **DESIGN OF GIRDERS.**—The girders are to be designed for a uniform live load of 125 lb. per sq. ft. of roadway. Allowing 30 per cent for impact, the live load for one girder is $1.30 \times 125 \times 8 = 1{,}300$ lb. per lin. ft.

The girder will be assumed as having a section 66 in. × 22 in. in calculating the dead load stresses, giving a load of 1,510 lb. per ft. of girder. The slab and wearing surface weigh 211 lb. per sq. ft. or $211 \times 8 = 1{,}690$ lb. per lin. ft., making a total dead load of $1{,}510 + 1{,}690 = 3{,}200$ lb. per lin. ft. per girder.
The maximum live load bending moment per girder and including 30 per cent impact is

$$M_L = \tfrac{1}{8} w \cdot l^2 = \tfrac{1}{8} \times 1{,}300 \times 35^2 = 199{,}600 \text{ ft.-lb.}$$

The dead load bending moment per girder is

$$M_D = \tfrac{1}{8} w \cdot l^2 = \tfrac{1}{8} \times 3{,}200 \times 35^2 = 490{,}000 \text{ ft.-lb.}$$

The total bending moment per girder is

$$M = 199{,}600 + 490{,}000 = 689{,}600 \text{ ft.-lb.} = 8{,}275{,}200 \text{ in.-lb.}$$

The depth to the center of the reinforcement, required by the unit stresses given in the specifications is

$$d = 0.0965 \sqrt{\frac{M}{b}} = 0.0905 \sqrt{\frac{8{,}275{,}200}{22}} = 59.25 \text{ in.} \tag{6c}$$

making a total depth of $59\tfrac{1}{4} + 2 + 3 = 64\tfrac{1}{4}$ in. if two layers of bars spaced 4 in. c. to c. are used, and the distance from the bottom of the beam to the center of the lower layer of bars is made 3 in. A total depth of 65 in. will be used, making the depth to the center of the steel $d = 60$ in. This section is so near the assumed section of 66 × 22 in. that no revision in the dead load moment will be made.

The area of steel required to develop this section is

$$A = 0.0077b\cdot d = 0.0077 \times 22 \times 60 = 10.2 \text{ sq. in.}$$

for the unit stresses given in the specifications. Eight $1\frac{1}{8}$-in. square bars provide an area of 10.20 sq. in. and will be used, and will be placed in two layers 4 in. apart c. to c. The bars may be spaced 5 in. c. to c. and allow $3\frac{1}{2}$ in. between the centers of the outside bars and the edges of the beam.

The live load end shear is

$$V_L = \tfrac{1}{2}w\cdot l = \tfrac{1}{2} \times 1{,}300 \times 35 = 22{,}750 \text{ lb.}$$

The dead load end shear is

$$V_D = \tfrac{1}{2}w\cdot l = \tfrac{1}{2} \times 3{,}200 \times 35 = 56{,}000 \text{ lb.}$$

The total end shear is

$$V = 22{,}750 + 56{,}000 = 78{,}750 \text{ lb.}$$

The maximum unit end shear is

$$f_v = \frac{V}{b\cdot j\cdot d} = 1.15\frac{V}{b\cdot d} = 1.15\frac{78{,}750}{22 \times 60} = 69 \text{ lb. per sq. in.} \qquad (68)$$

which requires that shear reinforcement be used. This will be calculated after the bond stress has been determined.

If the top layer of the tensile reinforcement be bent up to assist in carrying the shear, the maximum unit bond stress is

$$f_u = \frac{V}{\Sigma o\cdot j\cdot d} = 1.15\frac{V}{\Sigma o\cdot d} = 1.15\frac{78{,}750}{18 \times 62} = 82 \text{ lb. per sq. in.} \qquad (67)$$

the depth to the center of the lower layer of the bars being 62 in. The ends of these bars will be hooked to give additional bond strength.

In order to determine at what points bars may be bent up and still provide sufficient tensile reinforcement the bending moment diagram should be drawn, as shown in Fig. 24.

The equation for the bending moment at any point due to a uniform load is

$$M_x = \tfrac{1}{2}w\cdot l\cdot x - \tfrac{1}{2}w\cdot x^2 = \tfrac{1}{2}w(lx - x^2)$$

where $w$ = load per foot of length and $x$ = distance in ft. from the support to the point considered. The total load per foot is

$$w = 3{,}200 + 1{,}300 = 4{,}500 \text{ lb.}$$

when $x = 5$, $\quad M_x = 2{,}250\ (35 \times 5 - 25) = 341{,}000$ ft.-lb.

when $x = 10$, $\quad M_x = 2{,}250\ (35 \times 10 - 100) = 568{,}000$ ft.-lb.

when $x = 17.5$, $\quad M_x = 2{,}250\ (35 \times 17.5 - 17.5^2) = 689{,}600$ ft.-lb.

as previously calculated.

In order to calculate the stresses in the web reinforcement the shear diagram should be drawn as shown in Fig. 24.

The equation for the maximum shear at any point is

$$V_x = \frac{W_L}{2}(l - x)^2 + \frac{W_D}{2}(l - 2x)$$

$$W_L = 1{,}300; \qquad W_D = 3{,}200; \qquad l = 35 \text{ ft.}$$

when $x = 5$, $\quad V_x = \dfrac{1{,}300}{70} \times 30^2 + 1{,}600 \times 25 = 56{,}700$ lb.

$x = 10$, $\quad V_x = \dfrac{1{,}300}{70} \times 25^2 + 1{,}600 \times 15 = 35{,}640$ lb.

$x = 17.5$ $\quad V_x = \dfrac{1{,}300}{70} \times 17.5^2 + 1{,}600 \times 0 = 5{,}700$ lb.

The shear which may be carried by the concrete without exceeding 40 lb. per sq. in. is

$$V_c = f_v \cdot b \cdot j \cdot d = 40 \times 22 \times 0.875 \times 60 = 46{,}200 \text{ lb.}$$

which locates the point $A$ at which web reinforcement begins to be necessary. The web reinforcement is considered as taking $\frac{2}{3}$ of the total shear from this point to the end of the beam.

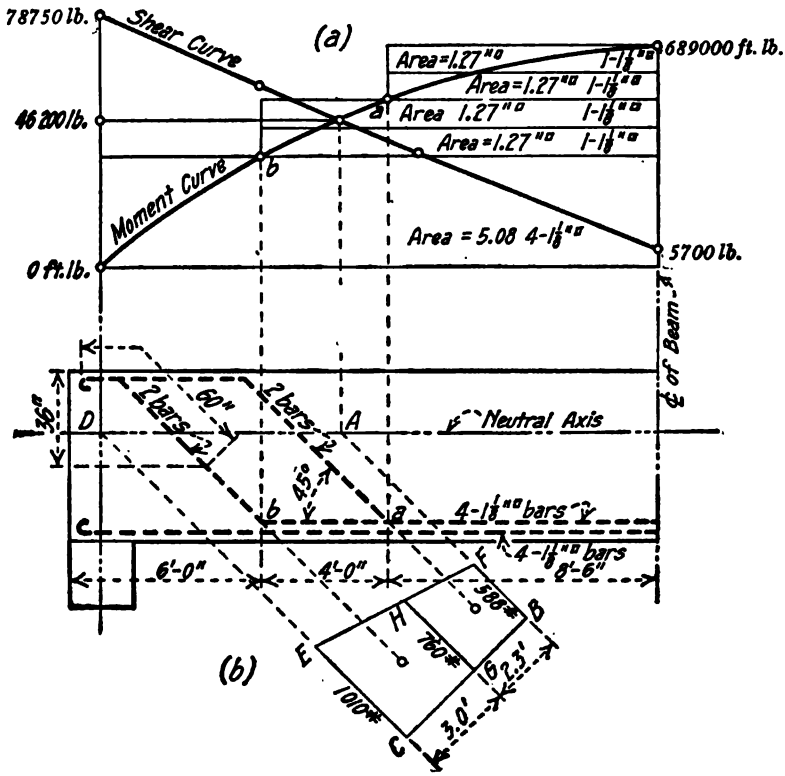



FIG. 24.

The stresses in the bent-up bars are determined by drawing $AB$ and $DC$ at the angle at which the bars are to be bent.

From $C$ lay off

$$CE = \frac{2}{3} f_v \cdot b = \frac{2}{3} \frac{V}{j \cdot d}$$

$$CE = \frac{2}{3} \frac{V}{j \cdot d} = \frac{2}{3} \times \frac{78{,}750}{0.874 \times 60} = 1{,}010 \text{ lb.}$$

and

$$BF = \frac{2}{3} \frac{V}{j \cdot d} = \frac{2}{3} \times \frac{46{,}200}{0.874 \times 60} = 588 \text{ lb.}$$

Draw $EF$.

It is evident from the moment diagram that two bars can be bent up at $a$ and two more at $b$. Bending the bars up at these points gives a good distribution. The line $GH$ is so drawn that the bars projected will pass as nearly as possible through the centroids of the corresponding areas.

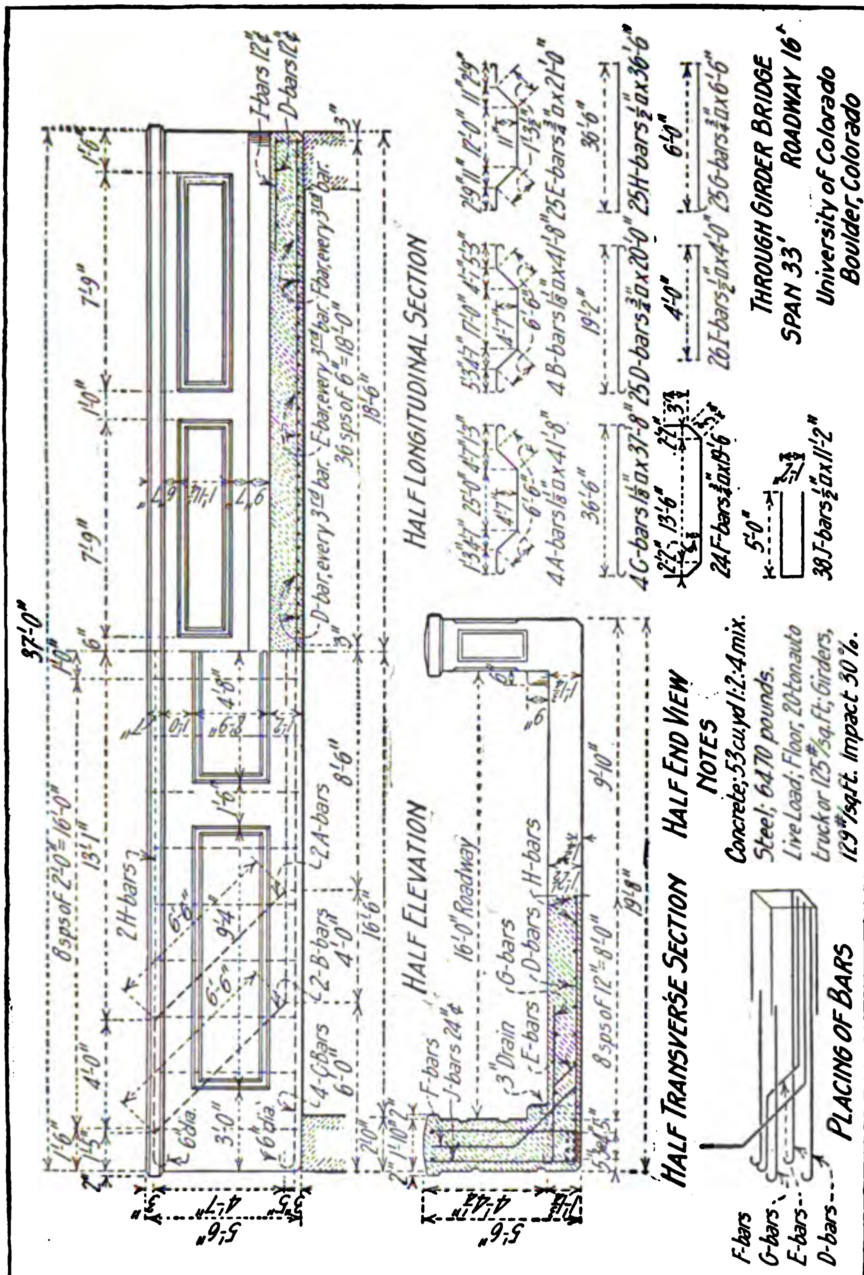

HALF ELEVATION
HALF LONGITUDINAL SECTION
HALF END VIEW
HALF TRANSVERSE SECTION
NOTES
Concrete; 53 cu.yd 1:2:4 mix.
Steel; 6470 pounds.
Live Load; Floor, 20-ton auto truck or 125#/sq.ft; Girders, 129#/sq.ft. Impact 30%.
PLACING OF BARS
F-bars
G-bars
E-bars
D-bars
THROUGH GIRDER BRIDGE
SPAN 33'
ROADWAY 16'
University of Colorado
Boulder, Colorado
16'-0" Roadway
3" Drain
37'-0"
4 C-bars 1 1/8"□x37'-8"
25 D-bars 3/4"□x20'-0"
25 H-bars 1/2"□x36'-6"
24 F-bars 3/4"□x19'-6"
26 I-bars 1/2"□x4'-0"
75 G-bars 3/4"□x6'-6"
38 J-bars 1/2"□x11'-2"
4 A-bars 1 1/8"□x41'-8"
4 B-bars 1 1/8"□x41'-8"
25 E-bars 3/4"□x21'-0"

Fig. 25.

The stress in the two bars nearest the end is given by the area *ECGH* which is

$$\frac{1,010 + 760}{2} \times 3.0 \times 12 = 32,000 \text{ lb.}$$

or 16,000 lb. per bar, which produces a unit stress of 16,000 ÷ 1.27 = 12,600 lb. per sq. in. in the bars towards the end. The stress in the bars towards the center of the girder is given by the area *GHFB* which is

$$\frac{760 + 588}{2} \times 2.3 \times 12 = 18,600 \text{ lb.}$$

or 9,300 lb. per bar, which produces a unit stress of 9,300 ÷ 1.27 = 7,300 lb. per sq. in.

The allowable stress in web reinforcement is 12,000 lb. per sq. in. so one set of bars is overstressed 600 lb. per sq. in. Vertical stirrups will be provided to take care of this excess and to make the beam more reliable.

The length of embedment required to develop the stress in the bent-up bars which are most severely stressed is

$$l = \frac{f_s \cdot d}{4f_u} = \frac{12,600 \times 1.125}{4 \times 80} = 45 \text{ in.}$$

Considering 0.6 the depth of the beam as effective in embedding the bar, the actual length of embedment is 60 in. as shown in Fig. 24.

6. **Detail Drawings.**—The detail drawings of this bridge are shown in Fig. 25. The bill of bars showing the number of bars and the dimensions for bending is included on this sheet.

# CHAPTER XXII.

## Design of Culverts.

**Types of Culvert.**—A culvert may be defined as a bridge of short span. The maximum limit of the span of a culvert is usually placed at 10 feet. Culverts may be divided into (1) pipe culverts; (2) box culverts; (3) bridge culverts; (4) arch culverts.

**DESIGN OF CULVERTS.**—The discharge of a culvert depends upon the size of the culvert, the grade of the culvert, the intake, the outlet, and the resistance of the barrel of the culvert to flow of water. Culverts will discharge very much more under a head than when flowing as an open conduit. The water should not be permitted to back up to give a head on the inlet of the culvert unless the embankment is water tight. Care should be taken in constructing a culvert to cut off the flow of water along the barrel of the culvert by means of projections and offsets.

**Size of Culverts.**—The area of waterway required for a culvert depends upon the maximum rate of rainfall, the size and shape of the watershed, the character of the soil and the grade and channel of the stream. Culverts placed in a "concrete dip," Chapter XXI are not designed for maximum run-off.

While various empirical formulas have been proposed, the size of waterway should always be checked by noting the efficiency of culverts in the vicinity. The best formula to use for the calculation of the size of culverts is Talbot's formula given in Chapter XX.

**Length of Culvert.**—The length of the barrel of the culvert is determined by the width of the roadway and height of the fill. The slope of the fill should be taken as 1 vertical on 1½ horizontal. The roadway should be the full width over culverts. The length of culvert required by the Iowa Highway Commission is given in Chapter IX.

**End Walls.**—The end walls may be flared with wings running back to take the fill; may have end walls parallel to the roadway, or where there is much drift the end walls may be stepped and run out parallel with the axis of the culvert. The drift will lodge on the steps but will not choke the culvert. The wing end walls make a better inlet and give a greater flow than the other types of end wall. Where there is danger from scour the culvert should have a floor.

**Pressure in Trenches.**—The pressures of the filling on pipes and other forms of culverts in trenches depends upon the character and condition of the fill, the shape and size of the trench, and the condition of the sides of the trench. The calculation of the pressures of the fill in trenches is practically the same problem as the calculation of the pressures in bins. The pressure in trenches may be calculated by using the formulas and data given in (7), Fig. 1. From this discussion it will be seen that the pressure will be decreased by laying the pipe in a narrow trench dug in the bottom of the main trench; the shoulders acting as bearing blocks to carry the fill.

A series of experiments on the pressures in trenches has been made at Iowa State College of Agriculture and Mechanic Arts, and are published in Bulletin No. 31, of the Experiment Station. The pressures obtained in the above experiments check with those given by the author's formula quite closely.

The analysis in (7), Fig. 1, assumes that the fill has recently been placed in a trench. With dry sand after final settlement the width of trench, *b*, should be taken equal to the outside diameter of the pipe. With clay with some cohesion the fill after final settlement will be partially self-supporting and the pressures will be decreased. For a concentrated load carried on the fill the load will be distributed over a width greater than the width of the concentrated load. The pressures will all be contained within an area made by the angle of repose of the fill with a vertical

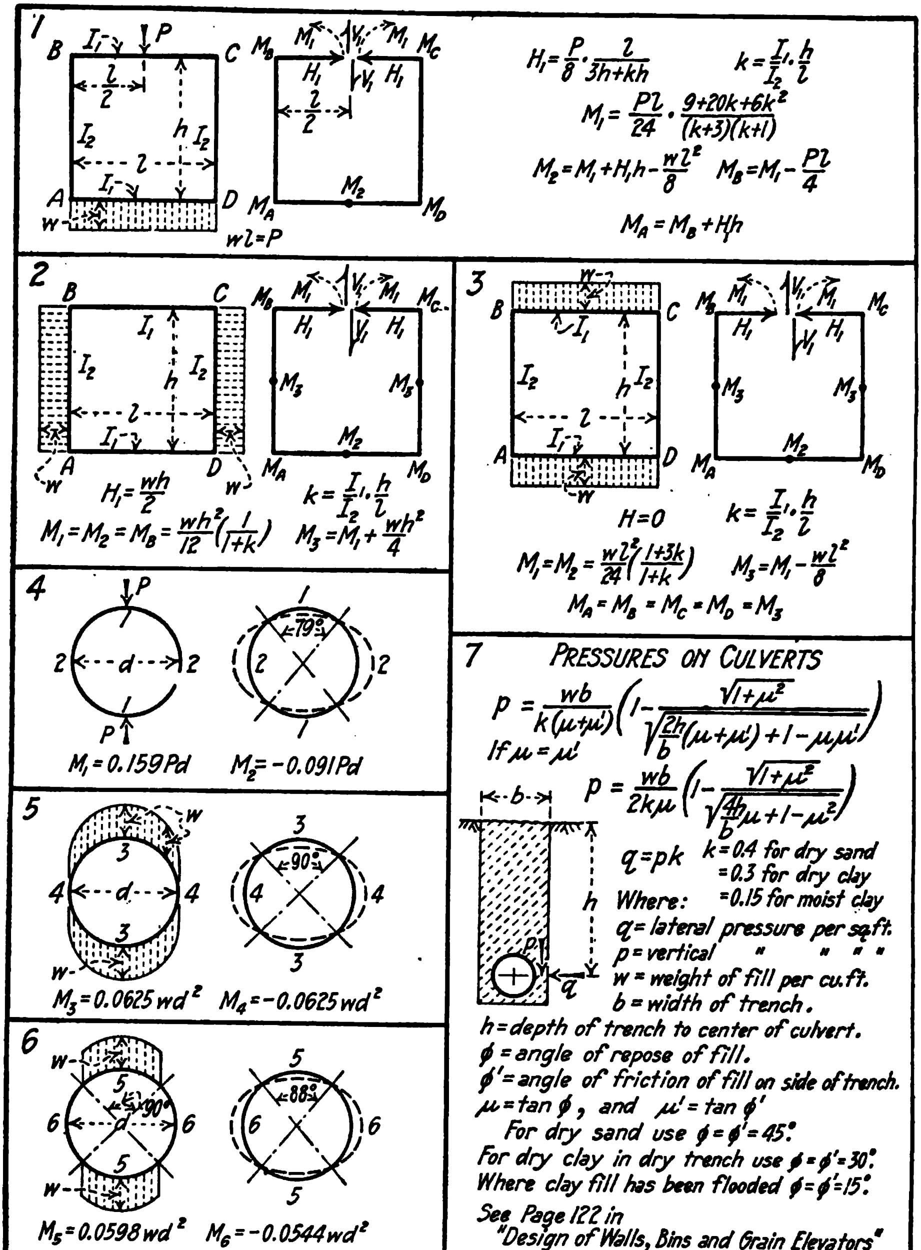
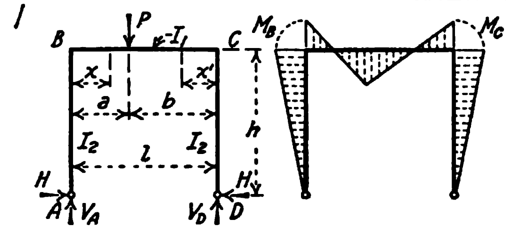



FIG. 1. STRESSES IN CULVERTS AND PRESSURES ON CULVERTS.

1



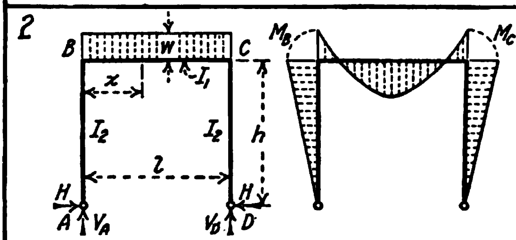

$$H=\frac{3Pab}{2hl(2k+3)} \qquad k=\frac{I_1}{I_2}\cdot\frac{h}{l}$$

$$V_A=\frac{Pb}{l} \qquad V_D=\frac{Pa}{l}$$

$$M_B=M_C=-Hh \qquad M_x=\frac{Pb}{l}x+M_B$$

$$M_{x'}=\frac{Pa}{l}x'+M_C \qquad M_{max}=\frac{4k+3}{2k+3}\cdot\frac{Pab}{2l}$$

2



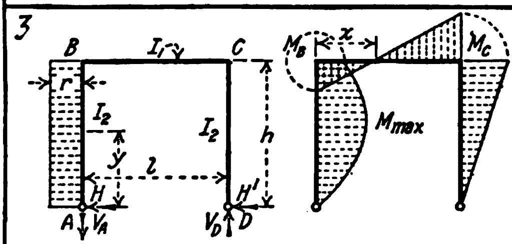

$$H=\frac{wl^2}{4h(2k+3)} \qquad k=\frac{I_1}{I_2}\cdot\frac{h}{l}$$

$$V_A=V_D=\frac{wl}{2} \qquad M_B=M_C=-Hh$$

$$M_x=\frac{wl}{2}x-\frac{wx^2}{2}+M_B$$

$$M_{max}=\frac{2k+1}{2k+3}\cdot\frac{wl^2}{8}$$

3




$$H=\frac{11k+18}{2k+3}\cdot\frac{rh}{8} \qquad k=\frac{I_1}{I_2}\cdot\frac{h}{l}$$

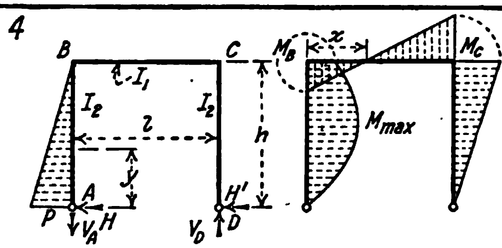
$$H'=rh-H \qquad V_A=-V_D=\frac{rh^2}{2l}$$

$$M_y=Hy-\frac{ry^2}{2} \qquad M_{max}=\frac{r}{2}\left(\frac{h}{8}\cdot\frac{11k+18}{2k+3}\right)^2$$

$$M_B=\frac{3rh^2}{8}\cdot\frac{k+2}{2k+3} \qquad M_C=-\frac{rh^2}{8}\cdot\frac{5k+6}{2k+3}$$

$$x=\frac{3l}{4}\cdot\frac{k+2}{2k+3} \qquad y=\frac{h}{8}\cdot\frac{11k+18}{2k+3}$$

4



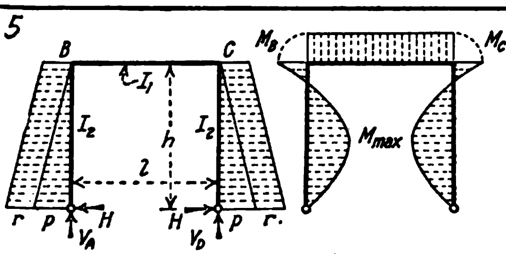

$$H=\frac{ph}{40}\cdot\frac{31k+50}{2k+3} \qquad k=\frac{I_1}{I_2}\cdot\frac{h}{l}$$

$$H'=\frac{ph}{2}-H=\frac{ph}{40}\cdot\frac{9k+10}{2k+3} \qquad V_A=-V_D=\frac{ph^2}{6l}$$

$$M_y=Hy-\frac{p}{6h}y^2(3h-y) \qquad M_B=\frac{ph^2}{120}\cdot\frac{13k+30}{2k+3}$$

$$M_C=-\frac{ph^2}{40}\cdot\frac{9k+10}{2k+3} \qquad M_x=\frac{l}{20}\cdot\frac{13k+30}{2k+3}$$

5




$$H=\frac{3rh}{4}\cdot\frac{k+2}{2k+3}+\frac{ph}{20}\cdot\frac{11k+20}{2k+3}$$

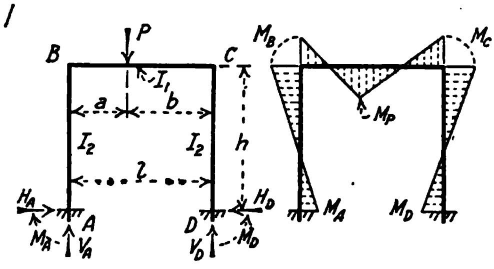
$$V_A=V_D=0 \qquad k=\frac{I_1}{I_2}\cdot\frac{h}{l}$$

$$M_B=M_C=-\frac{rh^2}{4}\cdot\frac{k}{2k+3}-\frac{ph^2}{60}\cdot\frac{7k}{2k+3}$$

$$M_{max}=-\frac{rh^2}{4}\cdot\frac{k}{2k+3}-\frac{ph^2}{60}\cdot\frac{7k}{2k+3}$$

FIG. 2. STRESSES IN RIGID FRAMES, PIN-CONNECTED AT BASE.

1




$$V_A = \frac{Pb}{l} \cdot \frac{1+S-2S^2+6k}{1+6k} \qquad k = \frac{I_1}{I_2} \cdot \frac{h}{l}$$

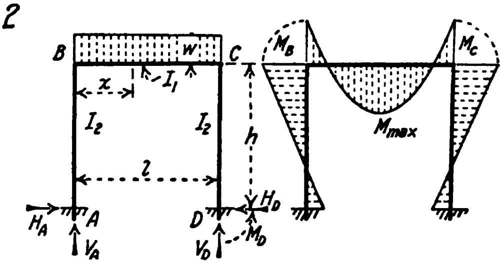
$$V_D = \frac{Pa}{l} \cdot \frac{3S-2S^2+6k}{1+6k} \qquad S = \frac{a}{l}$$

$$H_A = H_D = \frac{3Pab}{2hl(2+k)} \qquad M_A = \frac{abP}{2l} \cdot \frac{5k-1+2S(2+k)}{(2+k)(1+6k)}$$

$$M_D = \frac{abP}{2l} \cdot \frac{3+7k-2S(2+k)}{(2+k)(1+6k)} \qquad M_B = M_A - H_A h$$

$$M_C = M_D - H_D h \qquad M_P = M_A - H_A h + V_A a$$

2



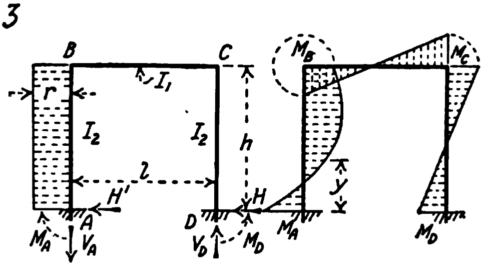

$$V_A = V_D = \frac{wl}{2} \qquad k = \frac{I_1}{I_2} \cdot \frac{h}{l}$$

$$H_A = H_D = \frac{wl^2}{4h(2+k)} \qquad M_A = M_D = \frac{wl^2}{12(2+k)}$$

$$M_B = M_C = -\frac{wl^2}{6(2+k)} \qquad M_x = \frac{wx}{2}(l-x) - \frac{wl^2}{6(2+k)}$$

$$M_{max} = \frac{wl^2}{24} \cdot \frac{2+3k}{2+k}$$

3




$$V_A = -V_D = \frac{rh^2k}{l(1+6k)} \qquad k = \frac{I_1}{I_2} \cdot \frac{h}{l}$$

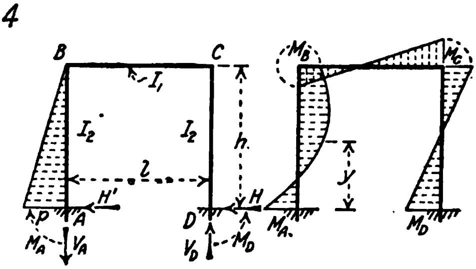
$$H = \frac{rh}{8} \cdot \frac{3+2k}{2+k} \qquad H' = rh - H$$

$$M_A = -\frac{rh^2}{24}\left(12 - \frac{9+5k}{2+k} - \frac{12k}{1+6k}\right)$$

$$M_D = \frac{rh^2}{24}\left(\frac{9+5k}{2+k} - \frac{12k}{1+6k}\right) \qquad M_B = M_A - Hh + \frac{rh^2}{2}$$

$$M_y = M_A + (rh-H)y - \frac{ry^2}{2} \qquad M_C = M_D - Hh$$

4




$$V_A = -V_D = \frac{pkh^2}{4l(1+6k)} \qquad k = \frac{I_1}{I_2} \cdot \frac{h}{l}$$

$$H = \frac{ph}{40} \cdot \frac{4+3k}{2+k} \qquad H' = \frac{ph}{2} - H$$

$$M_A = -\frac{ph^2}{120}\left(20 - \frac{12+7k}{2+k} - \frac{15k}{1+6k}\right) \qquad M_C = M_D - Hh$$

$$M_D = \frac{ph^2}{120}\left(\frac{12+7k}{2+k} - \frac{15k}{1+6k}\right) \qquad M_B = M_A - Hh + \frac{ph^2}{6}$$

$$M_y = M_A + \left(\frac{ph}{2} - H\right)y - \frac{py^2}{6h}(3h-y)$$

FIG. 3. STRESSES IN RIGID FRAMES, FIXED AT BASE.

plane. The pressures will decrease from the center to the outside. It is common to assume that the live load is carried down at an angle of one vertical to one-half horizontal, and is uniform on any horizontal section.

**Stresses in Circular Pipe.**—The stresses in circular pipe may be calculated by means of the formulas given in Fig. 1. The vertical and horizontal pressures are calculated at the center of the pipe, and are assumed as being equal to the pressures at the top and bottom of the pipe. The usual cases are

1. Single concentrated load, $P$, at top and bottom; $M = 0.159\ Pd$ under the loads, and $M = -0.091\ Pd$ at the ends of the horizontal diameter.

2. Uniform vertical load, $w$, over entire diameter, top and bottom; $M = 0.0625wd^2$ at middle of top and bottom, and $M = -0.0625\ wd^2$ at the ends of the horizontal diameter.

3. Uniform vertical load, $w$, over one-fourth the circumference, top and bottom: $M = 0.0598\ wd^2$ at middle of top and bottom, and $M = 0.0544\ wd^2$ at the ends of the horizontal diameter.

4. Uniform vertical load, $p$, over one-fourth the circumference top and bottom, and uniform horizontal load, $q$, over one-fourth the circumference on both sides and $M = +0.0598\ pd^2 - 0.0544\ qd^2$ at middle of top and bottom, and $M = 0.0598\ qd^2 - 0.0544\ pd^2$ at the ends of the horizontal diameter. Since the ratio, $k$, of lateral to vertical pressure, varies from 0.15 to 0.4, the lateral pressures do not materially reduce the pressures. Case 4, however, shows the importance of proper bedding of the pipe. Due to the uncertainty of the horizontal bedding, pipe culverts should be designed for vertical pressures only.

**Stresses in Box Culverts.**—The stresses in box culverts with a closed frame are given in Fig. 1. In the calculations $l$ = span of frame c. to c. side walls, and $h$ = height of frame c. to c. top and bottom walls. The top and bottom walls are assumed to have the same cross-section and moment of inertia.

Stresses in box culverts without a floorbeam and with lower ends of side walls free to turn, pin-connected, are given in Fig. 2. Case 3 represents the horizontal pressure due to a concentrated

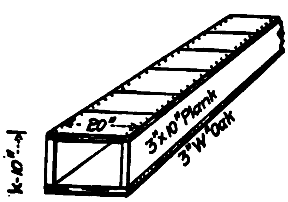



FIG. 4. TIMBER BOX CULVERT.

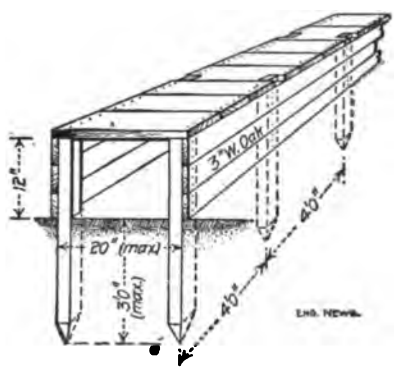



FIG. 5. TIMBER CULVERT.

load, which is assumed as a uniform horizontal load. In design the horizontal load should be assumed to act on both sides. Case 4 is for earth pressure as in a retaining wall, for which see Chapter XIX. In case 5, the effect of concentrated loads and earth pressure acting on both sides have been combined.

Stresses in box culverts without a floorbeam, but with the lower ends of the side walls fixed in place are given in Fig. 3. The stresses due to both concentrated loads and earth pressure on both sides may be obtained by adding the moments on both sides in 3 and 4, Fig. 3. Unless the footings are very rigid it is the best practice to assume that the side walls are free to turn at the base, and design for the larger stresses given in Fig. 2.

**Timber Culverts.**—For temporary culverts the timber box culvert, as shown in Fig. 4, or the timber culvert, as shown in Fig. 5, may be used. The bottom of the culvert in Fig. 5 should be paved to prevent scour. Unless care is used to carefully tamp the filling, the water will flow along the sides of both of the culverts shown. Timber culverts are very unsatisfactory and in the long run are very expensive.

**Pipe Culverts.**—Vitrified clay, cast iron, steel plate, corrugated steel and concrete pipes are used for culverts. Pipe culverts should be laid on a firm foundation to a careful grade. The center should be raised so that there will be no hollows in the pipe. Head walls, preferably of masonry or concrete should extend high enough to carry the fill, and should be carried down far enough to prevent the water from following along the outside of the pipe. The pipe should preferably be laid in concrete. The depth of cover required by the different types of culvert are given in Table IV, and Fig. 6.

TABLE I.

VITRIFIED CLAY AND REINFORCED CONCRETE CULVERT PIPE. U. S. BUREAU OF PUBLIC ROADS.

| Inside Diam., In. | Vitrified Clay. Thickness of Shell, In. | Vitrified Clay. Depth of Socket, In. | Vitrified Clay. Length of Section. Ft. In. | Vitrified Clay. Weight per Lin. Ft., Lb. | Concrete. Thickness of Shell, In. | Concrete. Reinforcement. Desired: Weight per Sq. Ft. | Concrete. Reinforcement. For Example. A. S. & W. Co.'s Style. | Concrete. Weight per Lin. Ft., Lb. |
|---|---|---|---|---|---|---|---|---|
| 12 | 1 | 3 | 2 6 | 50 | 2 | .4 lb. in 1 layer | No. 3 = .44 lb. per sq. ft. | 85 |
| 15 | 1¼ | 3 | 2 6 | 70 | 2¼ | .5 lb. in 1 layer | No. 2 = .51 lb. per sq. ft. | 120 |
| 18 | 1½ | 3 | 2 6 | 100 | 2½ | .6 lb. in 2 layers | No. 5 = .63 lb. per sq. ft. | 160 |
| 24 | 2 | 4 | 2 6 | 180 | 3 | .8 lb. in 2 layers | No. 4 = .80 lb. per sq. ft. | 260 |
| 30 | 2¼ | 4 | 3 0 | 290 | 3½ | 1.0 lb. in 2 layers | No. 25 = 1.01 lb. per sq. ft. | 365 |
| 36 | 2½ | 5 | 3 0 | 390 | 4 | 1.2 lb. in 2 layers | No. 42 = 1.20 lb. per sq. ft. | 500 |
| "Double Strength" Salt Glazed. | | | | | Usual length of section is 4 ft. Tongue and groove, taper or socket joints may be used. Weights given do not include weights of joints. | | | |

TABLE II.

CAST IRON CULVERT PIPE.

Standard Cast Iron water pipe, Class "A," which is the lighest weight. (American Waterworks Standard Specifications.)

| Inside Diam., In. | Area of Opening, Sq. Ft. | Thickness of Shell, In. | Depth of Socket, In. | Weight, Lb. Per Ft. | Weight, Lb. 12-Ft. Section. |
|---|---|---|---|---|---|
| 12 | 0.8 | .54 | 4 | 72.5 | 870 |
| 14 | 1.1 | .57 | 4 | 89.6 | 1,075 |
| 16 | 1.4 | .60 | 4 | 108.3 | 1,300 |
| 18 | 1.8 | .64 | 4 | 129.2 | 1,550 |
| 24 | 3.1 | .76 | 4 | 204.2 | 2,450 |
| 30 | 4.9 | .88 | 4½ | 291.7 | 3,500 |
| 36 | 7.1 | .99 | 4½ | 391.7 | 4,700 |
| 42 | 9.6 | 1.10 | 5 | 512.5 | 6,150 |
| 48 | 12.6 | 1.26 | 5 | 666.7 | 8,000 |

*Vitrified Clay Pipe Culverts.*—Vitrified clay pipe is made in single and double strength or culvert pipe. The double strength pipe should preferrably be used for culverts. The pipe should be laid in a trench rounded off to fit the pipe with the bells up stream. The joints should be

## TABLE III.

RIVETED AND CORRUGATED STEEL CULVERT PIPE. U. S. BUREAU OF PUBLIC ROADS.

| Inside Diam., In. | Area of Opening, Sq. Ft. | Riveted Steel. | | | | Corrugated Steel. | | |
|---|---|---|---|---|---|---|---|---|
| | | Thickness of Shell, In. | Rivets. | | Weight per Lin. Ft., Lb. | Thickness of Steel. | | Weight per Lin. Ft., Lb. |
| | | | Min. Diam., In. | Max. Pitch, In. | | Gage. | In. | |
| 12 | 0.8 | | | | | 16 | .0625 = 1/16 | 10 |
| 15 | 1.2 | | | | | 16 | .0625 = 1/16 | 12½ |
| 18 | 1.8 | 3/16 | 1/2 | 4 | 40 | 16 | .0625 = 1/16 | 15½ |
| 24 | 3.1 | 1/4 | 1/2 | 5 | 70 | 14 | .0781 = 5/64 | 24 |
| 30 | 4.9 | 1/4 | 1/2 | 5 | 85 | 14 | .0781 = 5/64 | 30 |
| 36 | 7.1 | 5/16 | 5/8 | 6 | 130 | 12 | .1094 = 7/64 | 36 |

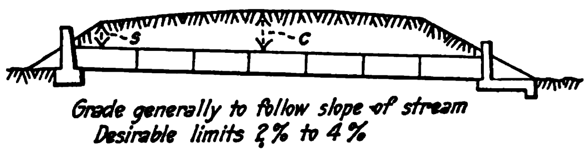



FIG. 6. MINIMUM DEPTH OF FILL ON PIPE CULVERTS. U. S. BUREAU PUBLIC ROADS. (See Table IV.

## TABLE IV.

MINIMUM DEPTHS OF GOOD FILL OR BALLAST OVER PIPE CULVERTS. U. S. BUREAU OF PUBLIC ROADS. FIG. 6.

| Inside Diam. | Cast Iron. | | Riveted Steel. | | Corrugated Metal. | | Vitrified Clay. | | Concrete. | | Inside Diam. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | C. | S. | C. | S. | C. | S. | C. | S. | C. | S. | |
| in. | ft. in. | in. | ft. in. | in. | ft. in. | ft. in. | ft. in. | ft. in. | ft. in. | in. | in. |
| 12 | 1 0 | 8 | 1 6 | 8 | 1 6 | 1 0 | 1 6 | 1 0 | 1 0 | 8 | 12 |
| 14 | 1 0 | 8 | | | | | | | | | 14 |
| 15 | | | 1 6 | 8 | 1 6 | 1 0 | 1 6 | 1 0 | 1 0 | 8 | 15 |
| 16 | 1 0 | 8 | | | | | | | | | 16 |
| 18 | 1 0 | 8 | 1 6 | 8 | 1 6 | 1 0 | 1 6 | 1 0 | 1 0 | 8 | 18 |
| 24 | 1 0 | 8 | 2 0 | 8 | 2 0 | 1 0 | 2 0 | 1 0 | 1 0 | 8 | 24 |
| 30 | 1 3 | 8 | 2 6 | 8 | 2 6 | 1 0 | 2 6 | 1 0 | 1 3 | 8 | 30 |
| 36 | 1 6 | 8 | 3 0 | 8 | 3 0 | 1 0 | 3 0 | 1 0 | 1 6 | 8 | 36 |
| 42 | 1 9 | 8 | | | | | | | | | 42 |
| 48 | 2 0 | 8 | | | | | | | | | 48 |

Note: If the material in the fill contains much clay, silt or loam increase the above minimum depths by 4 in.

calked with 1–3 Portland cement mortar. The earth should be well tamped around and over the pipe, and in no case should the wheels of wagons be permitted to come nearer to the top than the diameter of the pipe. Both ends of the pipe should be protected by masonry or concrete head walls as shown in Figs. 7 to 9.

The common sizes, weights and dimensions of vitrified clay pipe are given in Table I.

## TABLE V.

DIMENSIONS AND ESTIMATED QUANTITIES. STRAIGHT ENDWALLS FOR PIPE CULVERTS, FIG. 7.

| Dimensions. | | | | | | | | | | | | Concrete in One End Wall. | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Opening. | | Wall. | | | | | | Footing. | | | | 1:3:6 Mixture. | | | |
| | | | | | | | | | | | | Wall. | Footing. | Total. | |
| *D.* | Area, Sq. Ft. | *G.* | | *H.* | | *B.* | | *E.* | | *F.* | | Cu. Ft. | Cu. Ft. | Cu. Ft. | Cu. Yd. |
| in. | | ft. | in. | ft. | in. | ft. | in. | ft. | in. | ft. | in. | | | | |
| 12 | 0.8 | 4 | 0 | 2 | 0 | 1 | 2 | 1 | 10 | 1 | 0 | 7.2 | 7.3 | 14.5 | 0.54 |
| 15 | 1.2 | 5 | 0 | 2 | 3 | 1 | 2 | 1 | 10 | 1 | 2 | 9.9 | 10.7 | 20.6 | 0.76 |
| 18 | 1.8 | 6 | 0 | 2 | 6 | 1 | 3 | 1 | 11 | 1 | 3 | 13.6 | 14.4 | 28.0 | 1.04 |
| 24 | 3.1 | 8 | 0 | 3 | 0 | 1 | 4 | 2 | 0 | 1 | 4 | 22.3 | 21.3 | 43.6 | 1.62 |
| 30 | 4.9 | 10 | 0 | 3 | 6 | 1 | 6 | 2 | 2 | 1 | 6 | 34.7 | 32.5 | 67.2 | 2.49 |
| 36 | 7.1 | 12 | 0 | 4 | 0 | 1 | 8 | 2 | 4 | 1 | 8 | 50.5 | 46.7 | 97.2 | 3.60 |
| 42 | 9.6 | 14 | 0 | 4 | 6 | 1 | 10 | 2 | 6 | 2 | 0 | 70.3 | 70.0 | 140.3 | 5.20 |
| 48 | 12.6 | 16 | 0 | 5 | 0 | 2 | 1 | 2 | 9 | 2 | 0 | 96.9 | 88.0 | 184.9 | 6.85 |

## TABLE VI.

DIMENSIONS AND ESTIMATED QUANTITIES. PIPE CULVERT ENDWALLS WITH 45° WINGS, FIG. 8.

| Dimensions. | | | | | | | | | | | | Quantities in One End Wall. | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Opening. | | Wall. | | | | | | | | Footing. | | 1 : 3 : 6 Concrete. | | | | Steel Tie Rods. |
| | | | | | | | | | | | | Wall. | Footing. | Total. | | |
| *D.* | Area, Sq. Ft. | *H.* | | *G.* | | *L.* | | *M.* | | *F.* | | Cu. Ft. | Cu. Ft. | Cu. Ft. | Cu. Yd. | |
| in. | | ft. | in. | ft. | in. | ft. | in. | ft. | in. | ft. | in. | | | | | |
| 18 | 1.8 | 2 | 6 | 3 | 10 | 1 | 2 | 1 | 7 | 1 | 3 | 9.3 | 10.7 | 20.0 | .74 | none |
| 24 | 3.1 | 3 | 0 | 4 | 4 | 1 | 5 | 2 | 1 | 1 | 4 | 13.1 | 14.4 | 27.5 | 1.02 | 2, ¾″φ 2 ft. long |
| 30 | 4.9 | 3 | 6 | 4 | 10 | 1 | 9 | 2 | 5 | 1 | 6 | 17.4 | 18.8 | 36.2 | 1.34 | 2, ¾″φ 2 ft. long |
| 36 | 7.1 | 4 | 0 | 5 | 4 | 2 | 0 | 2 | 11 | 1 | 8 | 22.6 | 24.6 | 47.2 | 1.75 | 2, ¾″φ 3 ft. long |
| 42 | 9.6 | 4 | 6 | 5 | 10 | 2 | 3 | 3 | 6 | 2 | 0 | 29.1 | 34.6 | 63.7 | 2.36 | 2, ¾″φ 3 ft. long |
| 48 | 12.6 | 5 | 0 | 6 | 4 | 2 | 6 | 4 | 0 | 2 | 0 | 35.9 | 39.1 | 75.0 | 2.78 | 2, ¾″φ 3 ft. long |

## TABLE VII.

DIMENSIONS AND ESTIMATED QUANTITIES. PIPE CULVERT ENDWALLS WITH U-TYPE WINGS, FIG. 9.

| Dimensions. | | | | | | | | | | | | Quantities in One End Wall. | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Opening. | | Wall. | | | | | | Footing. | | | | 1:3:6 Concrete. | | | | Steel Tie Rods. |
| | | | | | | | | | | | | Wall. | Footing. | Total. | | |
| *D.* | Area, Sq. Ft. | *G.* | | *H.* | | *K.* | | *F.* | | *J.* | | Cu. Ft. | Cu. Ft. | Cu. Ft. | Cu. Yd. | |
| in. | | ft. | in. | ft. | in. | ft. | in. | ft. | in. | ft. | in. | | | | | |
| 12 | 0.8 | 3 | 8 | 2 | 0 | 1 | 0 | 1 | 3 | 2 | 2 | 6.6 | 7.3 | 13.9 | .52 | none |
| 15 | 1.2 | 3 | 11 | 2 | 3 | 1 | 5 | 1 | 3 | 2 | 7 | 8.3 | 9.1 | 17.4 | .64 | none |
| 18 | 1.8 | 4 | 2 | 2 | 6 | 1 | 9 | 1 | 3 | 2 | 11 | 9.9 | 10.7 | 20.6 | .76 | none |
| 24 | 3.1 | 4 | 8 | 3 | 0 | 2 | 6 | 1 | 6 | 3 | 8 | 13.9 | 15.5 | 29.4 | 1.09 | 2, ¾″φ 2 ft. long |
| 30 | 4.9 | 5 | 2 | 3 | 6 | 3 | 3 | 1 | 6 | 4 | 5 | 18.7 | 20.0 | 38.7 | 1.43 | 2, ¾″φ 2 ft. long |
| 36 | 7.1 | 5 | 8 | 4 | 0 | 4 | 0 | 1 | 9 | 5 | 2 | 24.2 | 26.2 | 50.4 | 1.87 | 2, ¾″φ 2½ ft. long |
| 42 | 9.6 | 6 | 2 | 4 | 6 | 4 | 9 | 2 | 0 | 5 | 11 | 30.3 | 33.2 | 63.5 | 2.35 | 2, ¾″φ 2½ ft. long |
| 48 | 12.6 | 6 | 8 | 5 | 0 | 5 | 6 | 2 | 0 | 6 | 8 | 37.3 | 39.6 | 76.9 | 2.85 | 2, ¾″φ 3 ft. long |

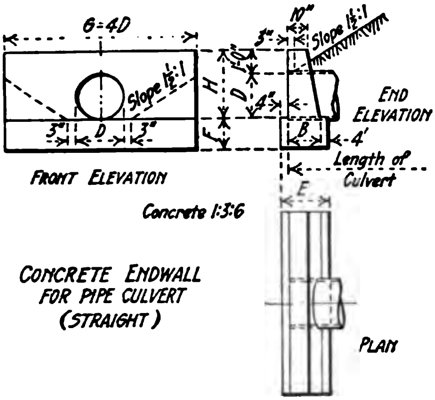



FIG. 7. STRAIGHT CONCRETE ENDWALL FOR CULVERTS.
U. S. BUREAU OF PUBLIC ROADS.

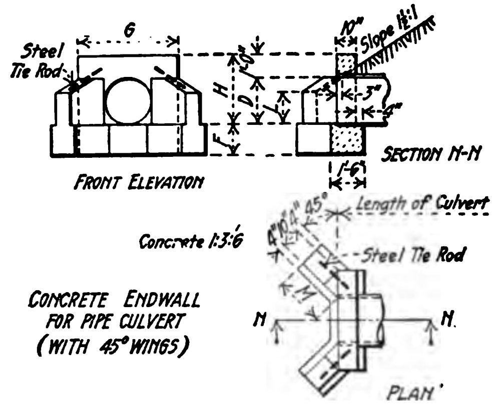



FIG. 8. CONCRETE ENDWALL FOR PIPE CULVERTS, WITH 45° WINGS.
U. S. BUREAU OF PUBLIC ROADS.

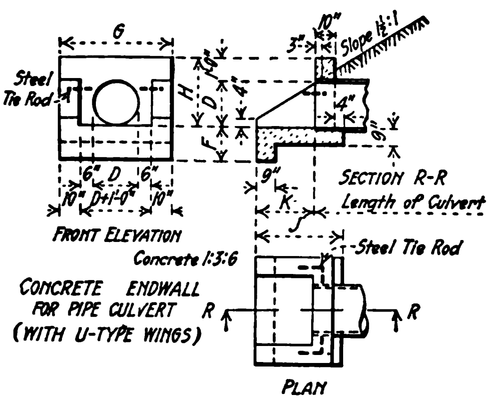



FIG. 9. CONCRETE ENDWALLS FOR PIPE CULVERTS, WITH U-TYPE WINGS.
U. S. BUREAU OE PUBLIC ROADS.

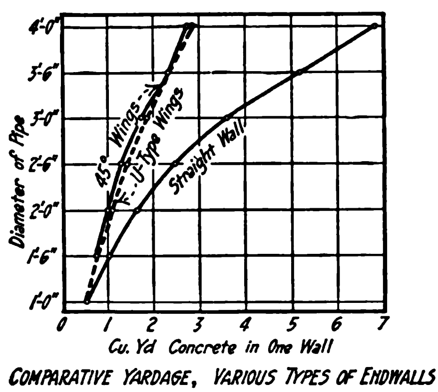



FIG. 10. CONTENTS OF ENDWALLS.
U. S. BUREAU OF PUBLIC ROADS.

*Cast Iron Pipe Culverts.*—Cast iron pipe for use in culverts can be obtained in 12-ft. lengths, in 3-ft. lengths, or in sectional form, the sections being bolted together in place. Cast iron pipe can be laid nearer the surface than vitrified clay pipe and is not damaged by freezing water. Cast iron pipe should be laid in the same manner as clay pipe and should have substantial end walls. Cast iron pipe is made with different weights per foot, the lighter weights being ordinarily used for culverts. The weight per foot for cast iron pipe is given in Table II.

*Steel Plate Pipe Culverts.*—Culverts are made of steel plates riveted in a circular or semicircular form. Plate pipe culverts should be laid with care and should have masonry head walls. The plates should be not less than $\frac{3}{16}$ in. thick for small sizes, and up to $\frac{3}{8}$ in. for culverts 4 feet in diameter and over, see Table III. The fill should extend above the top of the pipe a distance as given in Table IV, and should be well tamped around the sides. In estimating the cost of steel pipe culverts add 10 to 15 per cent to the weight of the plates to cover the laps and the rivets. The freight rates on culvert pipe are liable to be high, due to the fact that it is difficult to get sufficient weight on a car to give a minimum car load unless the pipes are of different sizes and can be nested. Data for riveted steel culvert pipe as prepared by the U. S. Bureau of Public Roads are given in Table III.

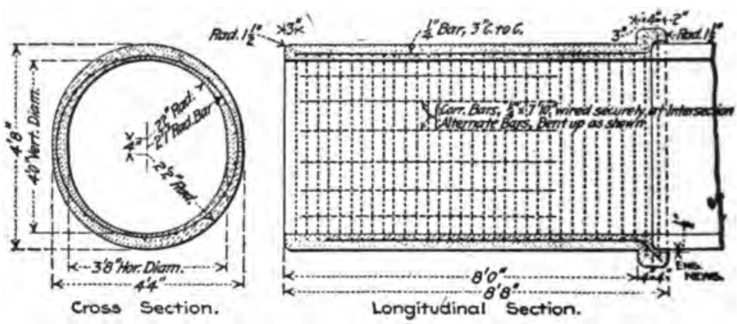



FIG. 11. REINFORCED CONCRETE CULVERT PIPE, C. B. & Q. R. R.

**Corrugated Steel Pipe.**—Culvert pipe made of corrugated metal sheets makes a very satisfactory culvert where the soil is not corrosive and where there is adequate cover over the pipe. The depth of cover required for corrugated metal culvert pipe is given in Table IV. Corrugated metal pipe is ordinarily made of structural steel containing a small percentage of copper, or of one of the so-called "pure irons." With data now available it would appear that the "pure irons" have no advantage in resisting corrosion over mild structural steel, and are inferior to high-grade copper steel.

*Specifications for Corrugated Metal Pipe.*—The Minnesota State Highway Department specifies that all corrugated metal pipe shall be either mild structural steel containing not less than two tenths per cent copper; or "ingot" or "pure" iron. All materials shall be galvanized with pure zinc coating of uniform thickness without imperfections. The coating of one square foot of plate on both sides shall contain not less than 1½ ounces nor more than 2¼ ounces of pure zinc. All metal shall be branded, (1) with name of manufacturer, (2) with name of brand, (3) with gage. Rivets shall be of same material as sheets and shall be galvanized. Rivets shall not be spaced closer than two diameters of rivet, shall be driven cold, and shall have full size heads. Corrugations shall not be less than 2⅔ in. nor more than 3 in. center to center, with a depth of ½ in. for 2⅔-in. and ⅝ in. for 3-in. corrugations. The ends of culverts shall be reinforced with a galvanized metal band, riveted to the culvert at intervals of 10 in. or less. This band shall be the equivalent of ⅜ in. × 1 in. for No. 16 gage metal, ⅜ in. × 1½ in. for No. 14 and No. 12 gage metal, and ½ in. × 1½ in. for No. 10 gage metal. The thickness of metal shall be not less than No. 16 gage for 18-in. diameter pipe and smaller; No. 14 gage for over 18-in. to 30-in. pipe; No. 12 gage for over 30-in. to 48-in. pipe; No. 10 gage for over 48-in. to 60-in pipe. Metal culverts over 48 in. shall not be used without special strengthening. All joints shall be lap joints. Longitudinal joints shall lap not less than 2 in., and shall have a rivet at each corrugation. Circumferential shop seams shall lap one corrugation and shall have rivets spaced not more than 10 in. Field joints shall be

same material as the pipe; shall be not less than 8 in. wide for pipe up to 30 in. diameter, and 11 in. wide for larger sizes. Field bands shall be provided with bands not less than 1 in. × ¼ in., with ½-in. bolts. All connections shall be of galvanized or otherwise suitably protected metal.

**Reinforced Concrete Pipe Culverts.**—Details of a reinforced concrete pipe for culverts, as designed by C. H. Cartlidge for the C. B. & Q. R. R., are shown in Fig. 11, while the forms for molding a similar pipe are shown in Fig. 12. The relative costs of cast iron and reinforced concrete pipe, as given by Mr. O. P. Chamberlain in Eng. News, Dec. 20, 1906, are shown in Table VIII.

The costs for reinforced concrete pipe are low and should probably be increased 50 to 100 per cent for single culverts in addition to the cost of making the forms.

TABLE VIII.

RELATIVE COSTS OF CAST IRON AND REINFORCED CONCRETE PIPE.

| Diameter, Inches. | Cast Iron Pipe. | | | Reinforced Concrete Pipe. | | |
|---|---|---|---|---|---|---|
| | Thickness, Inches. | Weight per Foot, Lb. | Cost per Foot. | Thickness, Inches. | Weight per Foot, Lb. | Cost per Foot. |
| 12 | 33/64 | 75 | $ 2.44 | 2 | 88 | $0.16 |
| 18 | 47/64 | 167 | 5.43 | 3 | 222 | 0.36 |
| 24 | 1 | 250 | 8.13 | 4¼ | 420 | 0.68 |
| 36 | 1⅛ | 450 | 14.63 | 4¾ | 676 | 1.10 |
| 48 | 1 7/16 | 725 | 23.50 | 6 | 1,131 | 1.83 |

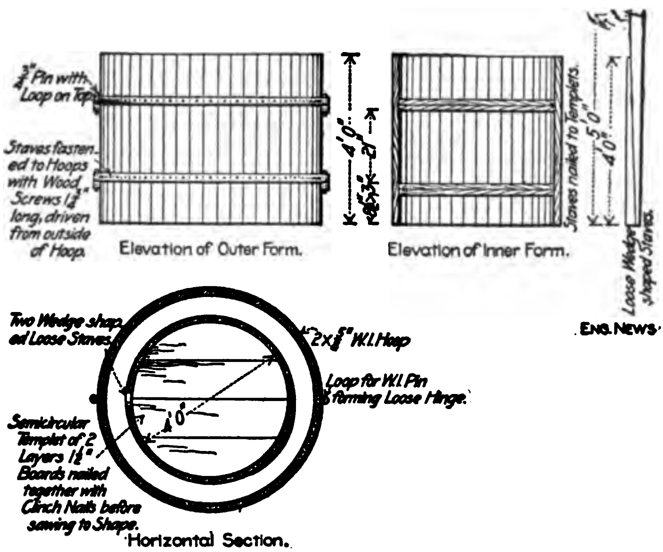



FIG. 12. FORMS FOR MOLDING REINFORCED CONCRETE CULVERT PIPE.

Forms for molding reinforced concrete culvert pipe are shown in Fig. 12.

Data for reinforced concrete culvert pipe as prepared by the U. S. Bureau of Public Roads are given in Table I.

Details of reinforced concrete pipe culverts designed by the Iowa Highway Commission are given in Fig. 13. Details of end walls with wing walls, and with walls parallel to the roadway are

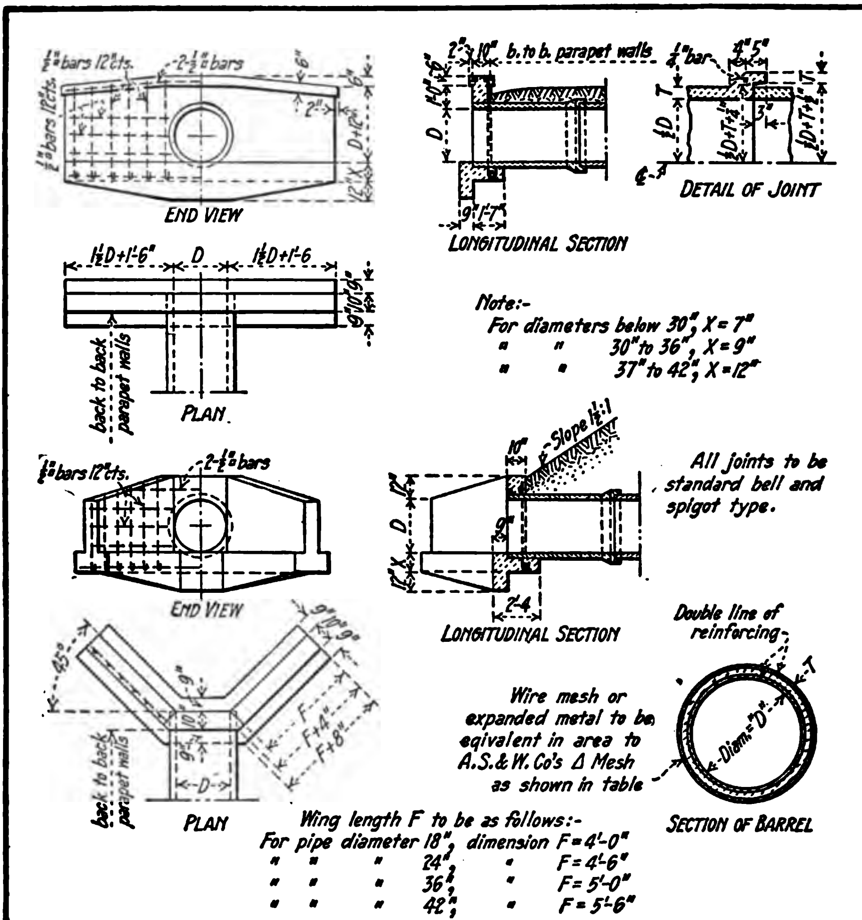



TABLE OF DIMENSIONS FOR CIRCULAR PIPE

| Diam. | Recommended Length | Weight | Thickness of shell = T | Area of steel for each line per 12" width |
|---|---|---|---|---|
| 15" | 5'-0" | 632 lb. | 2.25" | Style 6 = 0.058 sq. in. |
| 18" | | | 2.50" | " 5 = 0.077 " " |
| 24" | 4'-0" | 1173 lb. | 3.00" | " 4 = 0.102 " " |
| 30" | | | 3.50" | " 42 = 0.151 " " |
| 36" | 4'-0" | 2051 lb. | 4.00" | " 23 = 0.170 " " |
| 42" | | | 4.50" | " 32 = 0.225 " " |

CONCRETE PIPE CULVERTS
Iowa Highway Commission

FIG. 13.

shown in the cut. Wing walls give a better inlet than parallel end walls and are more economical when the culvert is at the bottom of a fill.

**Head Walls for Pipe Culverts.**—Head walls for pipe culverts may be made with straight end walls, Fig. 7, with wing walls as in Fig. 8, or with U-type wing walls as in Fig. 9. Data for straight end walls are given in Table V; for wing end walls are given in Table VI, and for U-type end walls are given in Table VII. The relative quantities of concrete in the three different types of head walls are given in Fig. 10. The plans of headwalls described above were prepared by the U. S. Bureau of Public Roads.

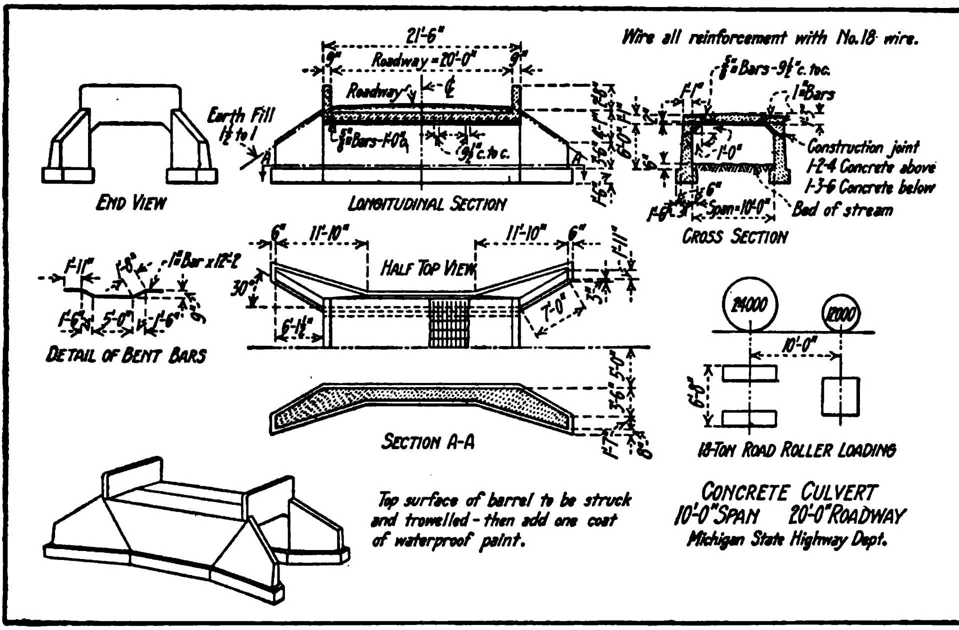



FIG. 14. REINFORCED CONCRETE CULVERT. MICHIGAN STATE HIGHWAY DEPARTMENT.

**Box Culverts.**—The box type of culvert is especially suited to a location where the top of the culvert is used as the top of the bridge, or where there is a shallow fill on top of the culvert. Box culverts when used without top filling are in fact short bridges. Box culverts may be used without a floor as in Fig. 14, or with a floor as in Fig. 15.

The box culvert shown in Fig. 14 was designed by the Michigan State Highway Department. Standard plans have been prepared for spans of 6 ft. to 15 ft. For spans under 6 ft. a reinforced concrete slab is supported on plain concrete abutments, the reinforcement at the upper corners being omitted.

Details of a reinforced concrete box culvert as designed by the Iowa Highway Commission are given in Fig. 15. Standard plans have been prepared by the commission for box culverts from 2 ft. by 2 ft. to 12 ft. by 12 ft. in cross-section. The design in Fig. 15, is for wing walls, but standards have been prepared for alternate designs with end walls parallel to the roadway. Data for standard box culverts with wing walls are given in Table IX.

**Reinforced Concrete Culverts.**—The plans of a reinforced concrete box culvert 4′ 0″ × 4′ 0″ are given in Fig. 16; of a reinforced concrete arch culvert 8′ 0″ × 8′ 0″ are given in Fig. 17;

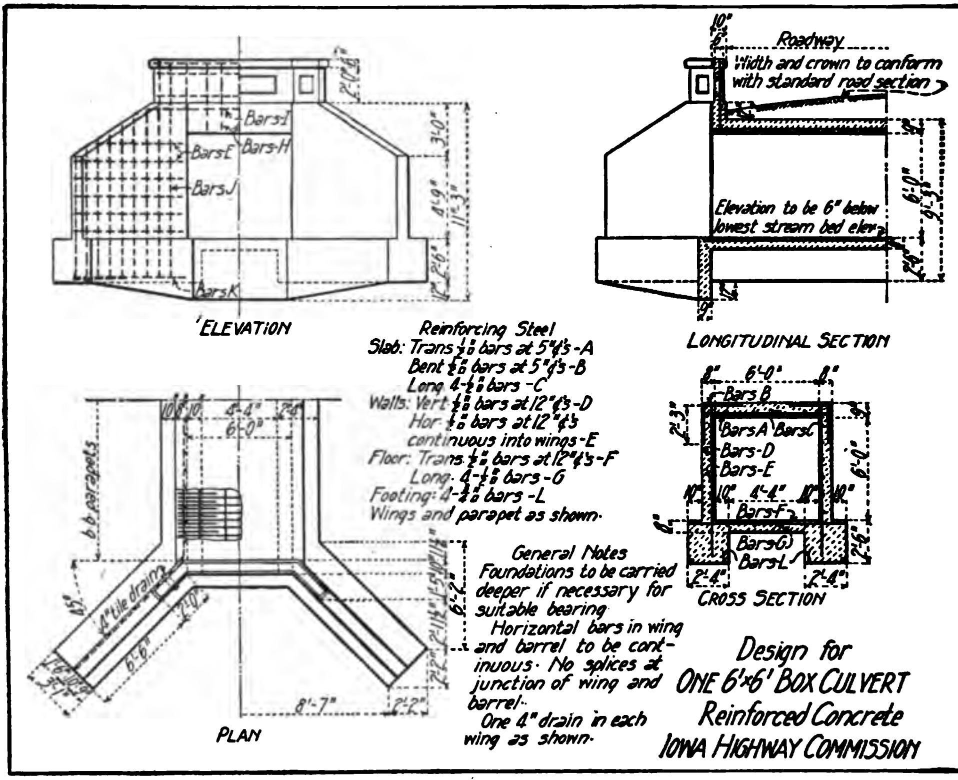



FIG. 15. CONCRETE BOX CULVERT. IOWA HIGHWAY COMMISSION.

## TABLE IX.

REINFORCED CONCRETE BOX CULVERTS. IOWA HIGHWAY COMMISSION.

| Size, Ft. | Depth of Sides, In. | Reinforcing, Square Bars. | | | | | | | | Quantities, 20-ft. Barrel with Wing Walls. | | Quantities, Change 1 Ft. Length. | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Slab A. | | Vertical Wall, D | | Floor Slab, F. | | Bent Bars, B. | | | | | |
| | | Size, In. | Spacing, In. | Size, In. | Spacing, In. | Size, In. | Spacing, In. | Size, In. | Spacing, In. | Concrete, Cu. Yd. | Steel, Lb. | Concrete, Cu. Yd. | Steel, Lb. |
| 2×2 | 6 | ½ | 12 | ½ in. sq. bars | 12 in. c. to c. | ½ in. sq. bars | 12 in. c. to c. | — | — | 8.0 | 542 | 0.23 | 20.3 |
| 3×3 | 6 | ½ | 10 | | | | | — | — | 13.5 | 754 | 0.39 | 25.7 |
| 4×4 | 7 | ½ | 8 | | | | | ½ | 8 | 24.0 | 1,385 | 0.60 | 46.7 |
| 5×5 | 8 | ½ | 6 | | | | | ½ | 6 | 32.2 | 1,840 | 0.83 | 61.6 |
| 6×6 | 8 | ½ | 5 | | | | | ½ | 5 | 40.4 | 2,344 | 1.04 | 79.2 |
| 8×8 | 10 | *¾ | 8 | | | | | Bars A Bent down. | | 73.0 | 2,856 | 1.58 | 91.0 |
| 10×10 | 12 | *¾ | 6 | | | | | | | 113.9 | 4,004 | 2.26 | 129.2 |
| 12×12 | 15 | *¾ | 5 | | | | | | | 163.4 | 6,924 | 3.16 | 198.0 |

* Every second bar bent up and turned down 2′ 6″ into side walls.

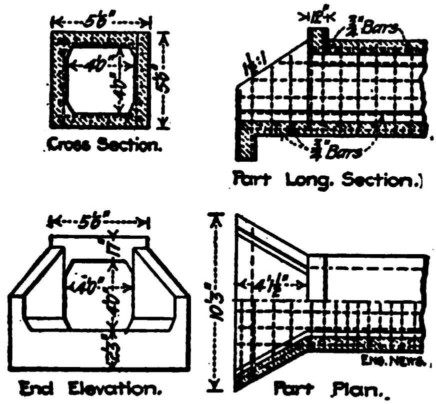



FIG. 16. REINFORCED CONCRETE BOX CULVERT, GREAT NORTHERN RAILWAY.

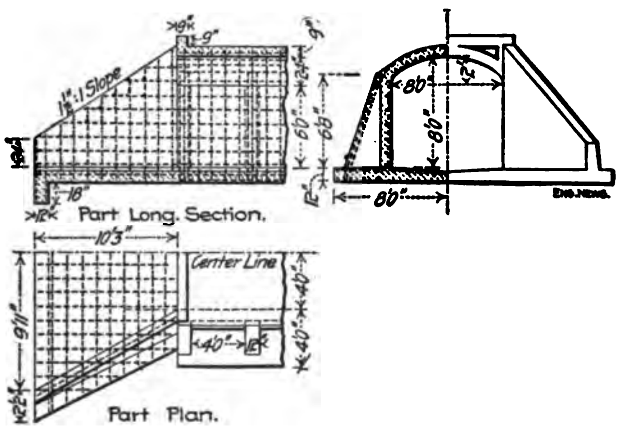



FIG. 17. REINFORCED CONCRETE ARCH CULVERT, GREAT NORTHERN RAILWAY.

## TABLE X.

RELATIVE COST OF SMALL CULVERTS OF APPROXIMATELY THE SAME WATERWAY, STANDARD SIZES.*

| Kind. | Size. | Area, Sq. In. of Water-way. | Cost Per Lineal Foot Laid. | Cost Per Sq. Ft. of Water-way. | Most Eco-nomical Length, Ft. | Cost of Culvert. | Cost of End Walls. | Cost Com-plete. | Cost to Keep up 100 Years. |
|---|---|---|---|---|---|---|---|---|---|
| 3 inch hemlock wood box | 15 in. sq. | 225 | $ .70 | $ .44 | 24 | $16.80 | ...... | $16.80 | $252.00 |
| Concrete box | 15 in. sq. | 225 | 1.10 | .70 | 20 | 22.00 | $18.00 | 40.00 | 40.00 |
| Concrete arch | 16 in. | 228 | 1.25 | .80 | 20 | 25.00 | 18.00 | 43.00 | 43.00 |
| Circular concrete pipe | 18 in. | 254 | .85 | .48 | 20 | 17.00 | 18.00 | 35.00 | 35.00 |
| Cast iron, 12 feet lengths | 18 in. | 254 | 3.10 | 1.76 | 24 | 74.40 | 18.00 | 92.40 | 166.80 |
| Cast iron, 3 feet lengths | 18 in. | 254 | 3.00 | 1.70 | 21 | 63.00 | 18.00 | 81.00 | 144.00 |
| Single strength vitrified clay | 18 in. | 254 | .90 | .51 | 30 | 27.00 | 14.00 | 41.00 | 41.00 |
| Double strength vitrified clay | 18 in. | 254 | 1.00 | .57 | 28 | 28.00 | 14.00 | 42.00 | 42.00 |
| Corrugated steel | 18 in. | 254 | 1.40 | .80 | 26 | 36.40 | 14.00 | 50.40 | 196.00 |

* A. R. Hirst, highway engineer, Wisconsin Geological and Natural History Survey, 1907.

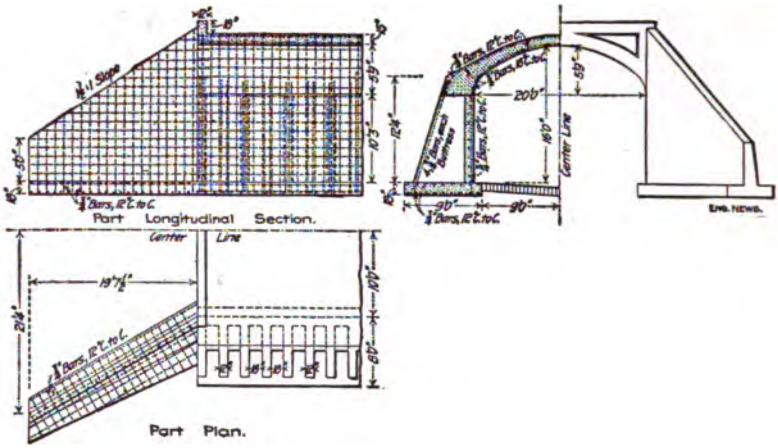



FIG. 18. REINFORCED CONCRETE ARCH CULVERT, GREAT NORTHERN RAILWAY.

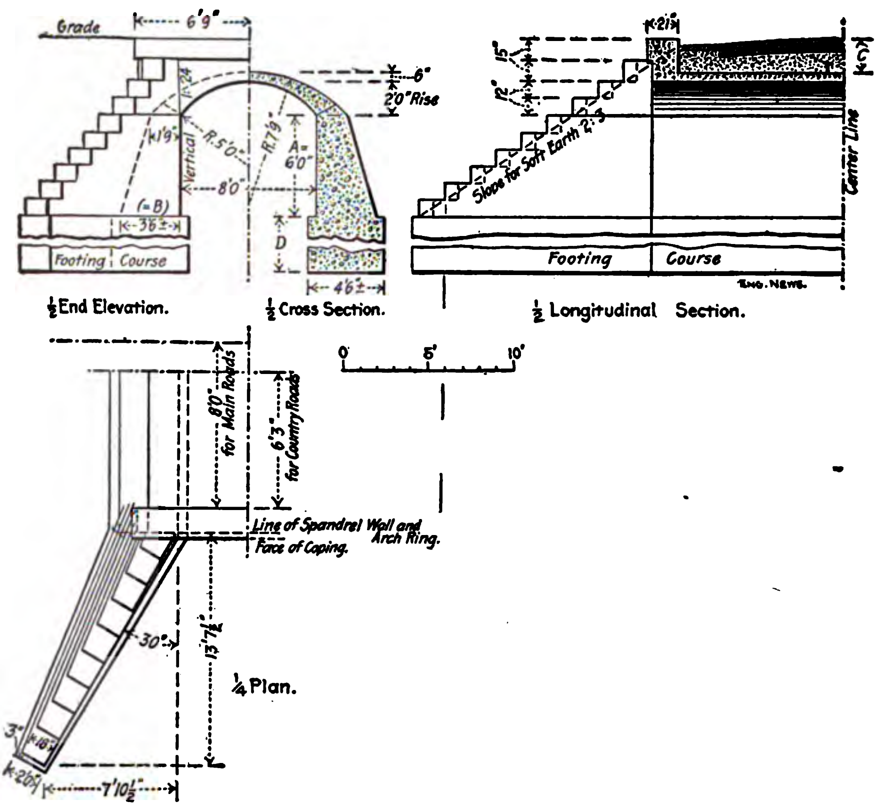



FIG. 19. PLANS OF EIGHT-FOOT PLAIN CONCRETE HIGHWAY CULVERT, PORTO RICO.

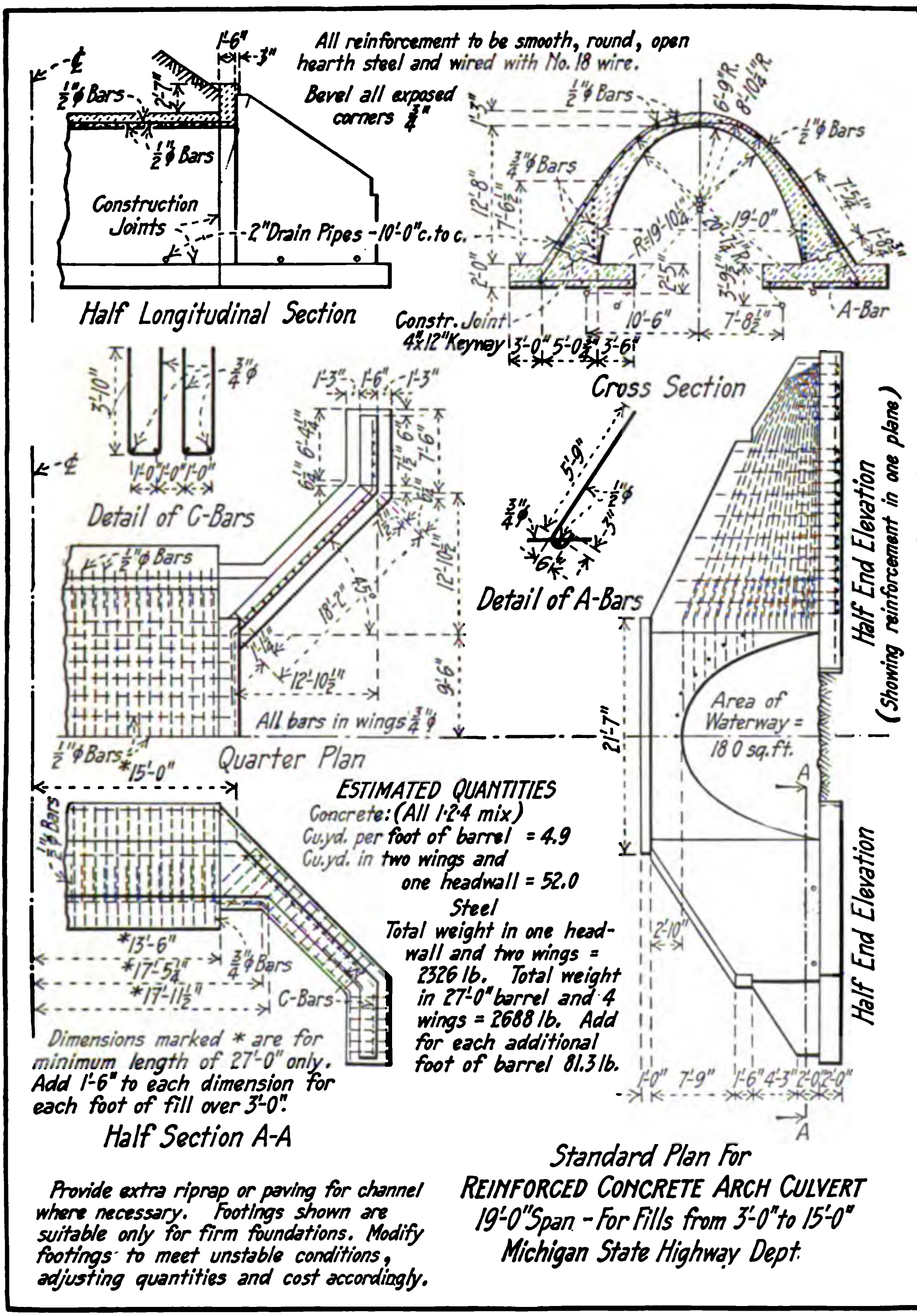



FIG. 20. REINFORCED CONCRETE ARCH CULVERT.

and of a reinforced concrete arch culvert of 20′ 0″ span are given in Fig. 18. These culverts were designed by Mr. C. F. Graff for the Great Northern Railway, and are strong enough to carry from 20 to 40 ft. of railroad embankment. These culverts are made heavier than is necessary for ordinary highway culverts.

**Plain Concrete Culverts.**—Plans for an 8-ft. plain concrete culvert, as designed by Edwin Thacher for the highways in Porto Rico, are shown in Fig. 19.

**Reinforced Concrete Arch Culvert.**—Details of a reinforced conrete arch culvert of 19 ft. span, as designed by the Michigan State Highway Department, are given in Fig. 20. Standards have also been prepared for arch culverts with straight end walls (0° with axis of stream). Details and data are shown in the cut.

**Relative Costs of Small Culverts.**—The relative costs of small culverts, as calculated by A. R. Hirst, highway engineer, Wisconsin Geological and Natural History Survey, are given in Table X.

CHAPTER XXIII.

## Design of Concrete Arch Bridges.

**Introduction.**—An arch is a beam or framework in which the reactions are not vertical for vertical loads. Arches are divided, according to the number of hinges, into three-hinged arches, two-hinged arches, one-hinged arches and arches without hinges or continuous arches. Solid two-hinged and continuous arches constructed of masonry or concrete, only, will be considered in this chapter. For the analysis of a two-hinged arch with spandrel bracing, see the author's "The Design of Steel Mill Buildings," Chapter XIV.

**DEFINITIONS.**—The following definitions will be of assistance in discussing arches.

**Skewback.**—The inclined surface upon which the arch rests. The term applies more properly to the stone or brick arch.

**Abutment.**—A skewback and the masonry which supports it.

**Soffit.**—The under or concave side of an arch.

**Back.**—The upper or convex side of an arch.

**Springing Line or Spring.**—The line in which the soffit meets the abutment. The inner edge of skewback.

**Intrados.**—The line of intersection of the soffit with a vertical plane parallel to the roadway.

**Extrados.**—The line of intersection of the back with a vertical plane parallel to the roadway.

**Span.**—The horizontal distance between springing lines, measured parallel to center line of roadway.

**Rise.**—The vertical distance of the intrados above a line joining the springing lines.

**Crown.**—The highest part of the arch ring.

**Haunch.**—The portion of the arch ring between the crown and the springing line.

**Spandrel.**—The space between the back of the arch and the roadway.

Two-hinged masonry or concrete arches are rarely used, but the theory of this type will be deduced as preliminary to the theory of the arch with fixed abutments.

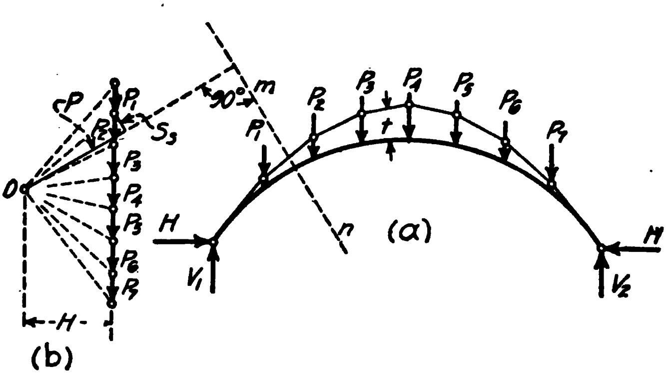



Fig. 1.

**STRESSES IN A TWO-HINGED ARCH. Reactions.**—The vertical reactions of the two-hinged arch in (a), Fig. 1, are the same as for a simple beam having the same loads and span. The horizontal reactions will be $H = H$, and will be equal to the pole distance of the force polygon in (b) that is used to draw the true equilibrium polygon. The value of $H$ depends upon the elasticity of the arch and is not statically determinate.

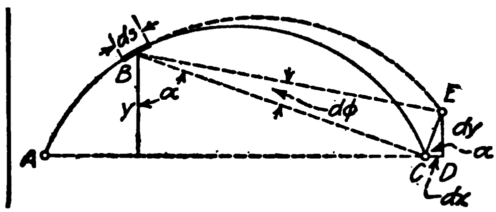



FIG. 2.

Having calculated the vertical reactions $V_1$ and $V_2$ by means of moments in the usual manner, and the horizontal components $H$, as will be described presently, the equilibrium polygon in (a) may be drawn using the force polygon in (b) Fig. 1. The requirements being that the equilibrium polygon must pass through the hinges, and that the force polygon must have a pole distance equal $H$.

The bending moment at any point in the arch will then be $H \cdot t$, where $H$ is the pole distance of the equilibrium polygon, and $t$ is the intercept from the point at which the moment is to be determined to the string $P_3P_4$. The shear on the section of the arch $m$–$n$ will be $S_s$, while the direct axial stress will be $P$, as shown in (b) Fig. 1.

**Calculation of Horizontal Reaction, H.**—Now for equilibrium in the two-hinged arch in Fig. 2, the span must remain constant. This relation will be expressed in the form of an equation of condition.

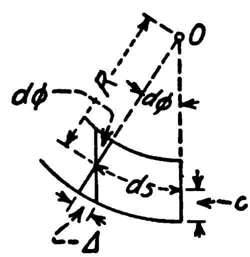



FIG. 3.

In Fig. 2 assume that the arch ring is rigid except the length $ds$ which bends under the action of some external loading. Now the point $C$ will move to $E$ if the arch be not constrained, the horizontal deformation being $CD = dx$, and the vertical deformation being $ED = dy$. The angle $CBE = d\phi$.

Then

$$
\begin{aligned}
CE &= BC \cdot d\phi \\
BC &= y \cdot \sec \alpha \\
dx &= CE \cdot \cos \alpha \\
&= y \cdot d\phi \qquad (1)
\end{aligned}
$$

Now in a beam, as in Fig. 3, the stresses at any point in the beam will vary as the distance from the neutral axis, and from similar triangles we have

$$R : ds :: c : \triangle$$

and

$$R \cdot \triangle = c \cdot ds \tag{2}$$

Now if $S$ is the fiber stress on the extreme fiber, and $E$ is the modulus of elasticity, we have $\triangle : S :: ds : E$, and

$$\triangle \cdot E = S \cdot ds \tag{3}$$

and solving (2) and (3) for $R$, we have

$$R \cdot S = E \cdot c \tag{4}$$

But from the common theory of flexure we have $M \cdot c = S \cdot I$, and substituting in (4)

$$R = E \cdot I / M \tag{5}$$

Also

$$R \cdot d\phi = ds, \text{ and } d\phi = ds/R \tag{6}$$

Substituting the value of $R$ as given in (5) in (6) we have

$$d\phi/ds = M/E \cdot I \tag{7}$$

And substituting the value of $d$ as given in (7) in (1)

$$dx = M \cdot y \cdot ds/E \cdot I \tag{8}$$

Now if bending takes place over the entire length of the span of the arch the total horizontal deformation will be

$$\Delta x = \int_0^l \frac{M \cdot y \cdot ds}{E \cdot I} \tag{9}$$

In the two-hinged arch the span is constant and

$$\Delta x = \int_0^l \frac{M \cdot y \cdot ds}{E \cdot I} = 0 \tag{10}$$

Equation (10) will be sufficient to determine the pole distance in Fig. 1.

Now in equation (10) the value of $M$ at any point in the arch will be $M = M' - H \cdot y$, where $M'$ is the bending moment as calculated in a simple beam, $H$ is the horizontal component of the reaction and $y$ is the ordinate of the point in the arch as in Fig. 2.

Inserting the value of $M$ in equation (10), it becomes

$$\int_0^l \frac{(M' - H \cdot y)}{E \cdot I} y \cdot ds = 0,$$

and

$$\int_0^l \frac{M' \cdot y \cdot ds}{E \cdot I} - H \int_0^l \frac{y^2 \cdot ds}{E \cdot I} = 0,$$

and

$$H = \frac{\displaystyle\int_0^l \frac{M' \cdot y \cdot ds}{E \cdot I}}{\displaystyle\int_0^l \frac{y^2 \cdot ds}{E \cdot I}} \tag{11}$$

**Graphic Solution.**—Now in Fig. 4 let polygon $AEB$ be a random polygon drawn with an assumed pole distance $H'$; polygon $ADB$ is the true equilibrium polygon drawn with the true pole distance $H$; and $ACB$ is the linear arch.

Then the bending moment at $C$ in Fig. 4 will be

$$\begin{aligned} M &= M' - H \cdot y \\ &= H \cdot CD \\ &= H \cdot DF - H \cdot CF \\ &= H \cdot DF - H \cdot y \end{aligned}$$

But $DF$ is not yet known. However, we have the relation that the ordinates to the two equilibrium polygons are inversely proportional to the pole distances; and

$$EF : DF :: H : H',$$

and

$$DF = EF \cdot H'/H = EF \cdot r,$$

where $r$ equals the ratio of the assumed pole distance to the true pole distance.

Then

$$M = EF \cdot H \cdot r - H \cdot y \tag{12}$$

Now substituting the value of $M$ given in (12) in (11), and

$$\int_0^l \frac{EF \cdot H \cdot r \cdot y \cdot ds}{E \cdot I} - \int_0^l \frac{H \cdot y^2 \cdot ds}{E \cdot I} = 0,$$

and

$$r = \frac{\displaystyle\int_0^l \frac{y^2 \cdot ds}{E \cdot I}}{\displaystyle\int_0^l \frac{EF \cdot y \cdot ds}{E \cdot I}} \tag{13}$$

Now in equation (13) if $E \cdot I$ is a constant, the arch may be divided into segments of equal length; or if $E \cdot I$ is not a constant the arch may be divided into segments for which $ds/E \cdot I$ is a constant, and we may write

$$r = \Sigma y^2 / \Sigma EF \cdot y \tag{14}$$

**Graphic Interpretation of Equations.**—Referring to Fig. 4, it will be seen that the numerator in (14) is the summation of the products of the ordinates to the arch taken at the centers of the

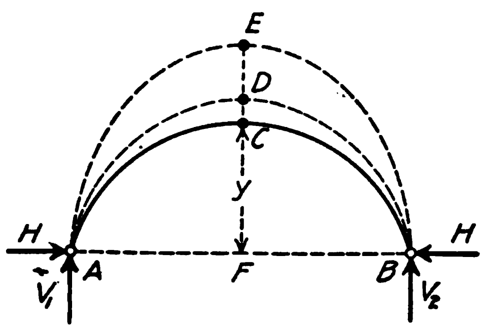



FIG. 4.

segments into which the arch ring is divided; while the denominator is the summation of the products of the ordinates to the random equilibrium polygon taken at the centers of gravity of the segments into which the arch ring is divided, and the distances of the segment from the line $AB$.

**Graphic Solution of the Stresses in a Two-hinged Arch.**—Divide the given arch ring into a number of segments, varying from 10 to 20 parts, in which $ds/E \cdot I$ equals a constant. Assume that the external loads act through the centers of gravity of the segments. Lay off the loads and construct a force polygon with a pole distance $H'$, and draw the equilibrium polygon so that it will pass through the hinges $A$ and $B$ of the arch. Now to use equation (13), scale off the ordinate $y$ of the center of each segment of the arch ring, and assume that these ordinates are horizontal loads acting at the points in the arch ring of which they are the ordinates; lay off these ordinates as horizontal loads and with the assumed pole distance $H''$ draw an equilibrium polygon with the force polygon thus constructed. In like manner assume that the ordinates to

the random equilibrium polygon at the corresponding points in the arch are horizontal forces; construct a force polygon with a pole distance $H''$ (use the same pole distance for both force polygons) and draw an equilibrium polygon with the force polygon thus constructed. The bending moments at the right abutment for the two loadings will be proportional to the horizontal deformation of the hinge $B$ for the two loadings, and $r$ will be equal to the ratio of the two bending moments as given by equation (13).

Having calculated the true pole distance, $H$, the true equilibrium polygon is drawn as in Fig. 1.

**Temperature Stresses.**—With an increase or decrease in temperature the arch will expand or contract uniformly and the change in the span will be

$$\Delta x = \pm e \cdot t \cdot l \tag{15}$$

where $e$ is the coefficient of linear expansion of the material ($e$ for steel and concrete is approximately 0.0000067 per degree Fahr.); $t$ is the change in temperature in degrees Fahr.; and $l$ is the span of the arch in the same units as $\Delta x$.

Then equation (10) becomes

$$\Delta x = \int_0^l \frac{M \cdot y \cdot ds}{E \cdot I} = \pm e \cdot t \cdot l \tag{16}$$

Now if $H_t$ is the horizontal component that will produce the same horizontal movement as the change in temperature as given by (16) then, $M = H_t \cdot y$, and

$$H_t = \pm \frac{e \cdot t \cdot l}{\int_0^l \frac{y^2 \cdot ds}{E \cdot I}} \tag{17}$$

The total value of the horizontal component of the reaction will be $H + H_t$.

**STRESSES IN AN ARCH WITHOUT HINGES. Introduction.**—In an arch without hinges the following conditions must be satisfied: (1) the span must be constant; (2) the abutments must maintain the same relative positions; and (3) the tangents to the neutral axis of the arch at the abutments must remain fixed.

From the discussion of the two-hinged arch we have

$$\Delta x = \int_0^l \frac{M \cdot y \cdot ds}{E \cdot I} = 0 \tag{10}$$

also in Fig. 2,

$$\begin{aligned} CE &= BC \cdot d\phi \\ BC &= x \cdot \csc \alpha \\ dy &= \text{C.E.} \cdot \sin \alpha \\ &= x \cdot d\phi \end{aligned}$$

and substituting in equation (7), $dy = M \cdot x \cdot ds / E \cdot I$, and

$$\Delta y = \int_0^l \frac{M \cdot x \cdot ds}{E \cdot I} = 0 \tag{18}$$

Also from equation (7), $d\phi = M \cdot ds / E \cdot I$, and

$$\Delta \phi = \int_0^l \frac{M \cdot ds}{E \cdot I} = 0 \tag{19}$$

The equations of condition for a fixed arch may also be obtained from the general equation given in text-books on applied Mechanics for the deflection of a beam at any point

$$\Delta = \int_0^l \frac{M \cdot m \cdot ds}{E \cdot I} \tag{20}$$

where $M$ is the bending moment at any point, and $m$ is the bending moment at any point due to a load unity applied at the point at which the deflection is to be measured, and acting in the line

that the deformation $\Delta$ is to be measured. Now if a unit load $H$ = unity be applied as in Fig. 5, the moment $m$ at any point will be $= y$, and

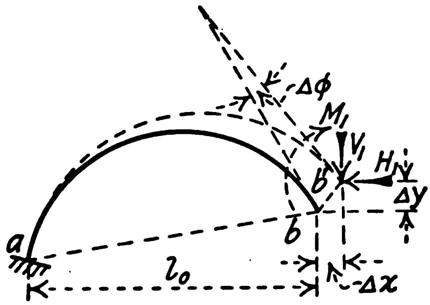



FIG. 5.

$$\Delta x = \int_0^l \frac{M \cdot y \cdot ds}{E \cdot I} = 0 \tag{10}$$

If a unit load $V$ = unity be applied as in Fig. 5, the moment $m$ at any point will be $= x$, and

$$\Delta y = \int_0^l \frac{M \cdot x \cdot ds}{E \cdot I} = 0 \tag{18}$$

If a unit moment be applied at any point, then $m = 1$, and

$$\Delta \phi = \int_0^l \frac{M \cdot ds}{E \cdot I} = 0 \tag{19}$$

Equation (10) is the condition that the span is constant; equation (18) that the abutments maintain the same relative heights, and equation (19) that the tangents to the neutral axis at the abutments are fixed.

In Fig. 6 at the left abutment, $A$, the vertical reaction is $V_1$, the horizontal reaction is $H$,

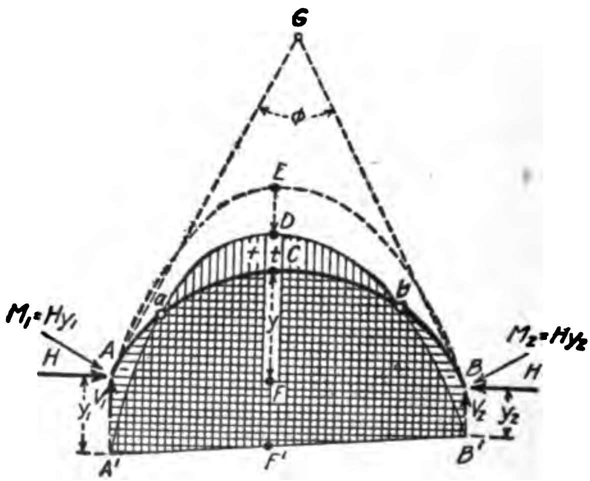



FIG. 6.

and the bending moment is $M_1 = H \cdot y_1$. At the right abutment, $B$, the vertical reaction is $V_2$, the horizontal reaction is $H$, and the bending moment is $M_2 = H \cdot y_2$. Now if the arch were hinged

at $A$ and $B$, the line of resistance would be the equilibrium polygon $AEB$ drawn with a force polygon having a pole distance equal to the horizontal reaction $H$ for a two-hinged arch. With the arch with fixed ends there is bending moment at points $A$ and $B$ and the equilibrium polygon will pass through points $A'$ and $B'$ and will take the position $A'DB'$, the points $a$ and $b$ being points of contra-flexure (points $A'$ and $B'$ may both be below the abutments, both above the abutments, or one may be below and the other above, depending upon the loading and elastic properties of the arch).

Now if the arch ring is divided into segments so that $ds/E \cdot I = g$, a constant, equations (10), (18) and (19) become

$$\Sigma M \cdot y = 0 \tag{10'}$$

$$\Sigma M \cdot x = 0 \tag{18'}$$

$$\Sigma M = 0 \tag{19'}$$

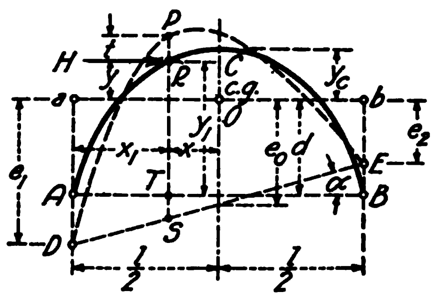



FIG. 7.

**Algebraic Solution.**—Transferring the origin of coordinates to the center of gravity of the arch ring, O, in Fig. 7, we have by substituting $y' = y + d$ in equations (19), (18) and (10)

$$\int_{-(l/2)}^{+(l/2)} M \frac{ds}{E \cdot I} = 0 \tag{19}$$

$$\int_{-(l/2)}^{+(l/2)} M \cdot x \frac{ds}{E \cdot I} = 0 \tag{18}$$

$$\int_{-(l/2)}^{+(l/2)} M \cdot y \frac{ds}{E \cdot I} = 0 \tag{10}$$

Now in Fig. 7 the bending moment

$$M = H \cdot t = H(PS - RS)$$

$$= M' - H \cdot y - H \cdot e_0 - H \frac{(e_1 - e_2)}{l} x$$

Now assume that

$$X = H \frac{(e_1 - e_2)}{l} = H \cdot \tan \alpha \tag{21}$$

and

$$Z = H \cdot e_0 = \tfrac{1}{2} H(e_1 + e_2) \tag{22}$$

then

$$M = M' - H \cdot y - X \cdot x - Z \tag{23}$$

Now substituting the value of $M$ in (23) in equations (10), (18), (19), respectively, and noting that due to symmetry

$$\int_{-(l/2)}^{+(l/2)} x \frac{ds}{E \cdot I} = 0, \tag{24}$$

and

$$\int_{-(l/2)}^{+(l/2)} x \cdot y \frac{ds}{E \cdot I} = 0, \tag{25}$$

and since we have chosen the $X-X$ axis so that

$$\int_{-(l/2)}^{+(l/2)} y \frac{ds}{E \cdot I} = 0 \tag{26}$$

we will have the following after reduction

$$H = \frac{\int_{-(l/2)}^{+(l/2)} \frac{M' \cdot y \cdot ds}{E \cdot I}}{\int_{-(l/2)}^{+(l/2)} \frac{y^2 \cdot ds}{E \cdot I}} \tag{27}$$

$$X = \frac{\int_{-(l/2)}^{+(l/2)} \frac{M' \cdot x \cdot ds}{E \cdot I}}{\int_{-(l/2)}^{+(l/2)} \frac{x^2 \cdot ds}{E \cdot I}} \tag{28}$$

$$Z = \frac{\int_{-(l/2)}^{+(l/2)} M' \cdot \frac{ds}{E \cdot I}}{\int_{-(l/2)}^{+(l/2)} \frac{ds}{E \cdot I}} \tag{29}$$

Now if the arch is divided into segments so that $ds/E \cdot I = g =$ a constant, the above equations will be

$$H = \frac{\Sigma M' \cdot y}{\Sigma y^2} \tag{30}$$

$$X = \frac{\Sigma M' \cdot x}{\Sigma x^2} = H \cdot \tan \alpha = (M_A + M_B)/l \tag{31}$$

$$Z = \frac{\Sigma M'}{n} = \tfrac{1}{2} H(e_1 + e_2) \tag{32}$$

where $n$ is the number of segments. Also the center of gravity is at a distance above the springing of

$$d = \frac{\Sigma y_1}{n} \tag{33}$$

Now let $V_A$ and $V_B$ be the end shears at $A$ and $B$, respectively; $M_A$ and $M_B$ be the bending moments at $A$ and $B$, respectively, and $R_A$ and $R_B$ be the reactions as for a simple beam at $A$ and $B$, respectively.

From equations (21) and (22)

$$e_1 = \tfrac{1}{2} X \cdot l/H + Z/H \tag{34}$$

$$e_2 = -\tfrac{1}{2} X \cdot l/H + Z/H \tag{35}$$

From Fig. 7

$$M_A = H(e_1 - d) \tag{36}$$

and

$$M_B = H(d - e_2) \tag{37}$$

also from (36), (37) and (21)

$$X = (M_A + M_B)/l \tag{38}$$

Take a section to right of $A$, and take moments about $B$, and

$$V_A \cdot l + M_A + M_B = R_A \cdot l$$

$$V_A = R_A - (M_A + M_B)/l$$

$$= R_A - X \tag{39}$$

also

$$V_B = R_B + X \tag{40}$$

**Temperature Stresses.**—With an increase or decrease in temperature the arch will expand or contract uniformly if there is no resistance. The tangents at the abutments will remain fixed which requires that

$$\int_{-(l/2)}^{+(l/2)} \frac{M \cdot ds}{E \cdot I} = 0. \tag{41}$$

The abutments will remain at the same relative heights, which requires that

$$\int_{-(l/2)}^{+(l/2)} \frac{M \cdot x \cdot ds}{E \cdot I} = 0. \tag{42}$$

Now if there is no constraint and the arch is free to move under the load

$$\Delta x = \int_{-(l/2)}^{+(l/2)} \frac{M \cdot y \cdot ds}{E \cdot I} = e \cdot t \cdot l \tag{43}$$

where $e$ is the coefficient of expansion = 0.000,006,7, $t$ = the change in temperature in degrees F., and $l$ = span of arch.

Now if movement is prevented, the horizontal reaction, $H_t$, will be obtained by substituting the value of $M$ given in equation (23) in equation (43) noting that $M' = 0$, and

$$H_t = \frac{e \cdot t \cdot l}{\int_{-(l/2)}^{+(l/2)} y^2 \frac{ds}{E \cdot I}} \tag{44}$$

also

$$X = 0 \text{ and } Z = 0.$$

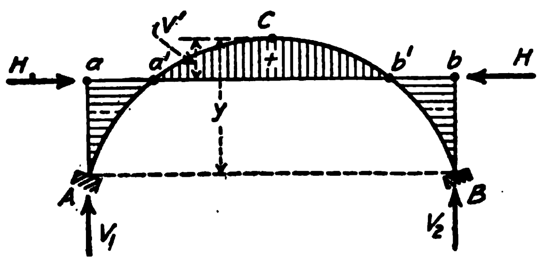



FIG. 8.

The horizontal thrust may be assumed to act through the center of gravity of the arch (point of contra-flexure) and the moments in the arch due to temperature will be as shown in Fig. 8.

**Stresses Due to Rib Shortening.**—The direct thrust will cause a shortening of the arch rib in addition to the stresses already calculated. If the direct thrust were constant for all sections of the arch ring the effect would be approximately the same as for a decrease in temperature. If $e$ is the coefficient of linear expansion per degree F., the change in temperature that will have the same shortening effect as the direct thrust will be

$$t = P/(A \cdot E \cdot e) \tag{45}$$

where $P$ = direct thrust = $H$ approximately, $A$ = average area of section and $E$ = modulus of elasticity.

The effect of rib shortening may also be calculated as follows: The work of deformation in the arch is

$$W = \int_0^l \frac{M^2 \cdot ds}{2E \cdot I} + \int_0^l \frac{N^2 \cdot ds}{2E \cdot A_1} \tag{46}$$

where $N$ is the normal stress on any section, $A_1$ is cross-section of the arch ring, and other symbols are as previously defined. Differentiating equation (46) with reference to $H$, $V$, and $M$, and substituting the values of $X$ and $Z$, we will have after reduction

$$H = \frac{\int_{-(l/2)}^{+(l/2)} \frac{M' \cdot y \cdot ds}{E \cdot I}}{\int_{-(l/2)}^{+(l/2)} \frac{y^2 \, ds}{E \cdot I} + \frac{s}{A \cdot E}} \tag{47}$$

where $A$ = average cross-section of the arch ring, and $s$ = length of the arch ring, also $X$ and $Z$ are as in equations (28) and (29), respectively. The value of $H$ in (47) differs from the value of $H$ in (27) only by the value $s/A \cdot E$ in the denominator. This value is relatively small except for light arches.

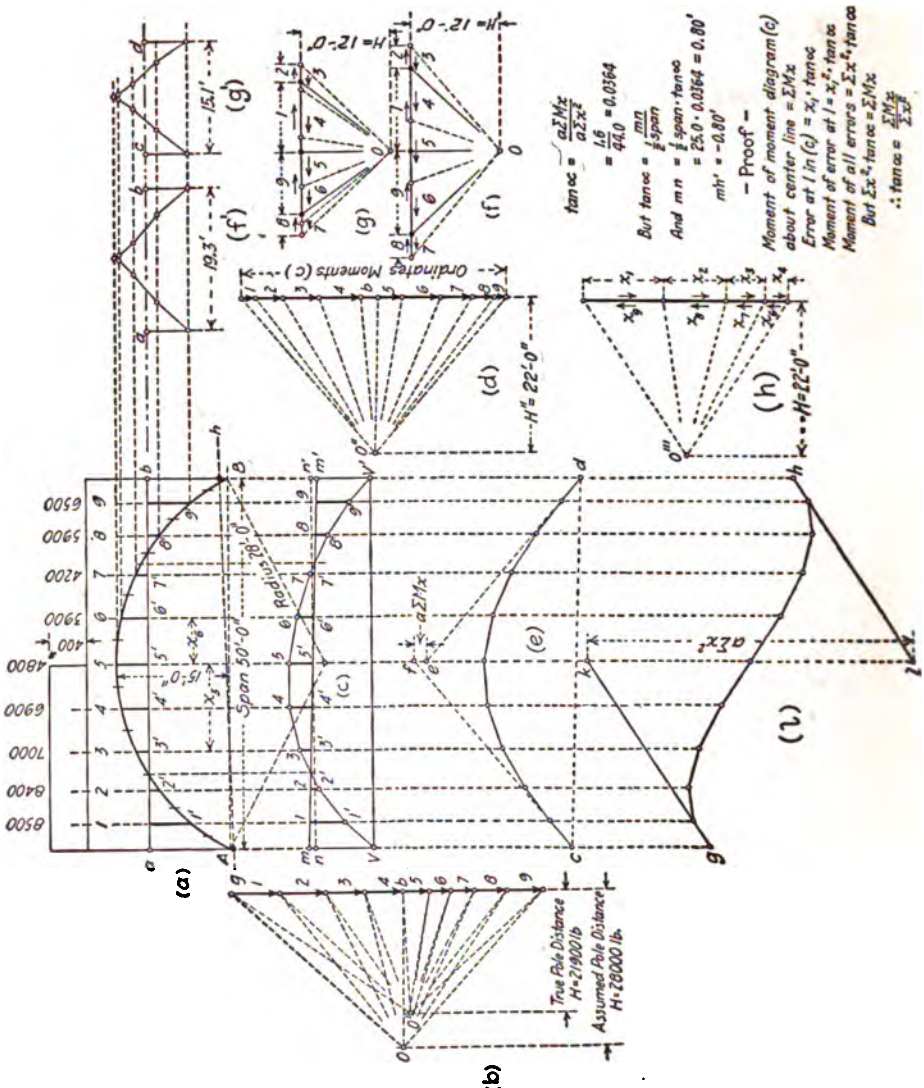



FIG. 9.

**Problem 1.**—Given a segmental reinforced concrete highway arch having a span of 50′ 0″, a rise of 15′ 0″, and a thickness of 2′ 0″, carrying a spandrel loading as shown and a live load of 400 lb. per square foot. The solution will be made for a live load over one-half the span. The arch ring will be divided into 10 equal segments, beginning and closing with a half segment as shown; the loads were calculated and numbered 1, 2, 3, 4, etc., as shown in Fig. 9.

Now lay off the loads 1, 2, 3, etc., and construct a force polygon with an assumed pole distance of 28,000 lb. as in (*b*). With force polygon in (*b*) construct equilibrium polygon $V5V'$ in (*c*). Now draw $nn'$ parallel to $VV'$, $nV$ being made equal to the arithmetical mean of the ordinates to the equilibrium polygon $V5V'$, at the points 1, 2, 3, etc. The sum of the ordinates 1–1′, 2–2′, 3–3′, etc., between $nn'$ and the equilibrium polygon $V5V'$ will now be equal to zero.

The angle $\alpha$ between the line $nn'$ and $mm'$ may be calculated by means of equation (31). Lay off the bending moments at points 1, 2, 3, etc. in (*c*), Fig. 9, in (*d*), Fig. 9 assume a pole distance = 22 ft., and construct equilibrium polygon in (*e*). Also lay off values of $x$, measured from the center of the arch, in (*h*) and draw equilibrium polygon in (*l*). Then $\tan\alpha = a\Sigma M'\cdot x/a\Sigma x^2 = 0.0312$ and $m-n = -m'-n' = 0.8$ ft. To calculate $H$, lay off values of the bending moments at 1–1′, 2–2′, 3–3′, etc. in (*g*) and with a pole = 12 ft., draw equilibrium polygon (*g′*), which gives an intercept 15.1 ft., and would be proportional to $\Sigma M'\cdot y$ if the true value of $H$ had been assumed. (The ordinates for $M'$ may be measured to the line $V-V'$ or to the parallel line $m-m'$.) Also lay off values of $y$ at points 1, 2, 3, etc. in (*a*) in (*f*), and with a pole distance = 12 ft. (pole distance for force polygons (*f*) and (*g*) should be equal but may have any convenient values), and draw equilibrium polygon (*f′*). Then intercept = 19.3 ft. and is proportional to $\Sigma y^2$.

Now the true value of $H$ will be found by the equation

$$28{,}000 \text{ lb.} : H :: 19.3 : 15.1$$

and

$$H = 21{,}900 \text{ lb.}$$

To draw the true equilibrium polygon in Fig. 10, draw $a-b$ as in Fig. 9, and lay off $a-V = m-v$ in (*c*) Fig. 10 $= m-v \times 28{,}000/21{,}900$ as given in (*c*), Fig. 9, also lay $b-V' = m'-v'$ in (*c*), Fig. 10 $= m'-v' \times 28{,}000/21{,}900$ as given in (*c*), Fig. 10. Now lay off the loads in force polygon (*b*), and with pole distance $H = 21{,}900$, draw equilibrium polygon beginning at $V$ and closing at $V'$. The position of pole $O$ in a vertical line in (*b*) may be determined as given in (*b*), Fig. 9. The pole $O$ in (*b*), Fig. 10, may be taken at any convenient point in a vertical plane at a distance $H = 21{,}900$ lb. from the load line, and equilibrium polygon (*c*) may be drawn. The true equilibrium polygon in (*a*) may be constructed by placing line $m-m'$ on $a-b$. The values 1–1′, 2–2′. 3–3′, etc., in (*c*) when multiplied by $H$ are the true bending moments at the corresponding points,

The shear at the section $gh$ = 2,000 lb. The direct thrust in the arch ring at the section $gh$ is $P = 32{,}000$ lb. The eccentricity of $P$ at the section $gh$ is $t = 4$ in., and the bending moment will be $M = 32{,}000 \times 4 = 128{,}000$ in.-lb.

The maximum stress on the section $gh$ is

$$S = P/A \pm M\cdot c/I$$

$$S = \frac{32{,}000}{24 \times 12} \pm \frac{128{,}000 \times 12}{\frac{1}{12} \times 12 \times 24^3}$$

$$= +111 \pm 111 = +222 \text{ or } 0 \text{ lb.}$$

**Temperature Stresses.**—Now for a change of ± 40 degrees F.

$$e\cdot t\cdot l = 0.000{,}006{,}7 \times 40 \times 600 \text{ in.}$$
$$= 0.16 \text{ in.}$$

The value of $ds/E\cdot I = g = (6 \times 12)/(2{,}000{,}000 \times 13{,}824)$

$$= 1/384{,}000{,}000$$

also

$$\Sigma y^2 = \tfrac{1}{2} \times 19.3 \times 12 \times 144$$
$$= 16{,}680$$

since the values of $y$ in (*f*) were laid off to twice the scale, and all dimensions are in inches.

Then

$$H_t = \pm \frac{0.16}{16{,}680 \times \dfrac{1}{384{,}000{,}000}} = \pm\, 3{,}760 \text{ lb.}$$

The temperature stresses do not change the vertical position of the pole $O$ in (*b*), Fig. 10, but the true pole for direct load and temperature will be, $H = 25{,}660$ lb. or $18{,}140$ lb.

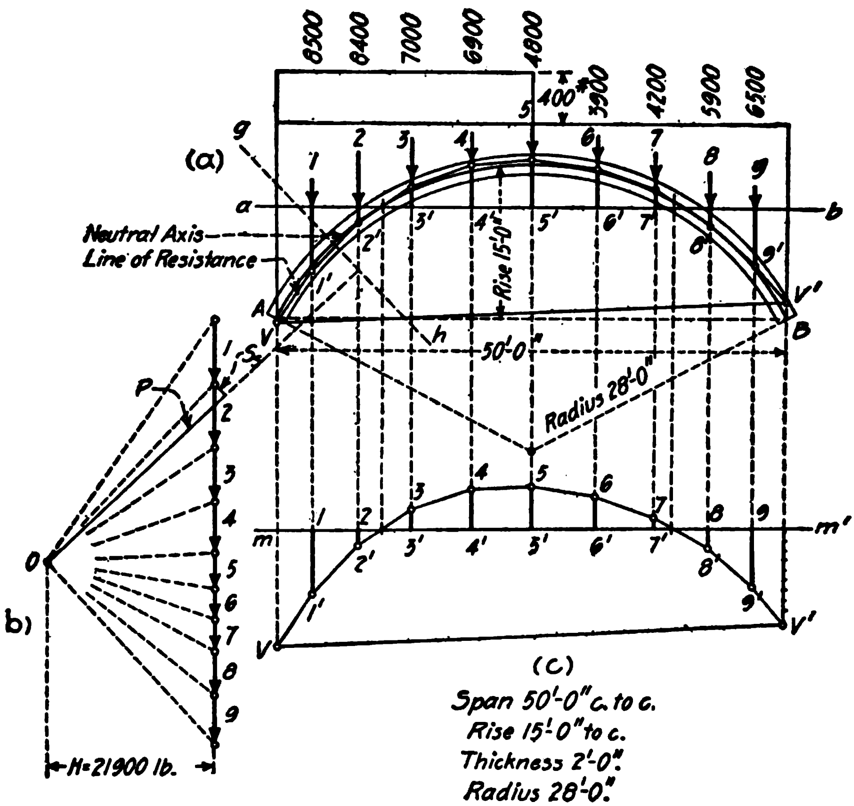



FIG. 10.

**Stresses Due Rib Shortening.**—The value of $P$ is approximately 32,000 lb., and from (45)

$$t = 32{,}000/(12 \times 24 \times 2{,}000{,}000 \times 0.000{,}006{,}7)$$

$$= -\,8.4°$$

The value of $H_t$ for this decrease in temperature will be $-\,8.4 \times 3{,}760/40 = -\,760$ lb.

**Summary.**—To provide for the stresses due to direct loads, the temperature and the direct thrust the pole distance will be $H = +\,21{,}900 \pm 3{,}760 - 760 = 24{,}900$ or $17{,}380$ lb. To complete the analysis the stresses in the arch should be calculated in the same manner as in Fig. 9 for a value of $H = 24{,}900$ lb. and $H = 17{,}380$ lb.

**INFLUENCE DIAGRAMS.**—The equations for $H$, $X$, and $Z$ for a unit load may be obtained from equations (30), (31), and (32), respectively, by substituting the bending moment for a unit load. For a unit load the moment $M'$ at any point in the arch will be the distance $x'$ from the load to the point in question, multiplied by unity $= x'$. Substituting $M' = x'$ in equations (30), (31) and (32) we have

$$H = \frac{\Sigma x' \cdot y}{\Sigma y^2} \tag{48}$$

$$X = \frac{\Sigma x' \cdot x}{\Sigma x^2} \tag{49}$$

$$Z = \frac{\Sigma x'}{n} \tag{50}$$

The influence lines for $H$, $X$ and $Z$ may be calculated by substituting in equations (48), (49), and (50), respectively, or may be calculated by graphics as in Fig. 11.

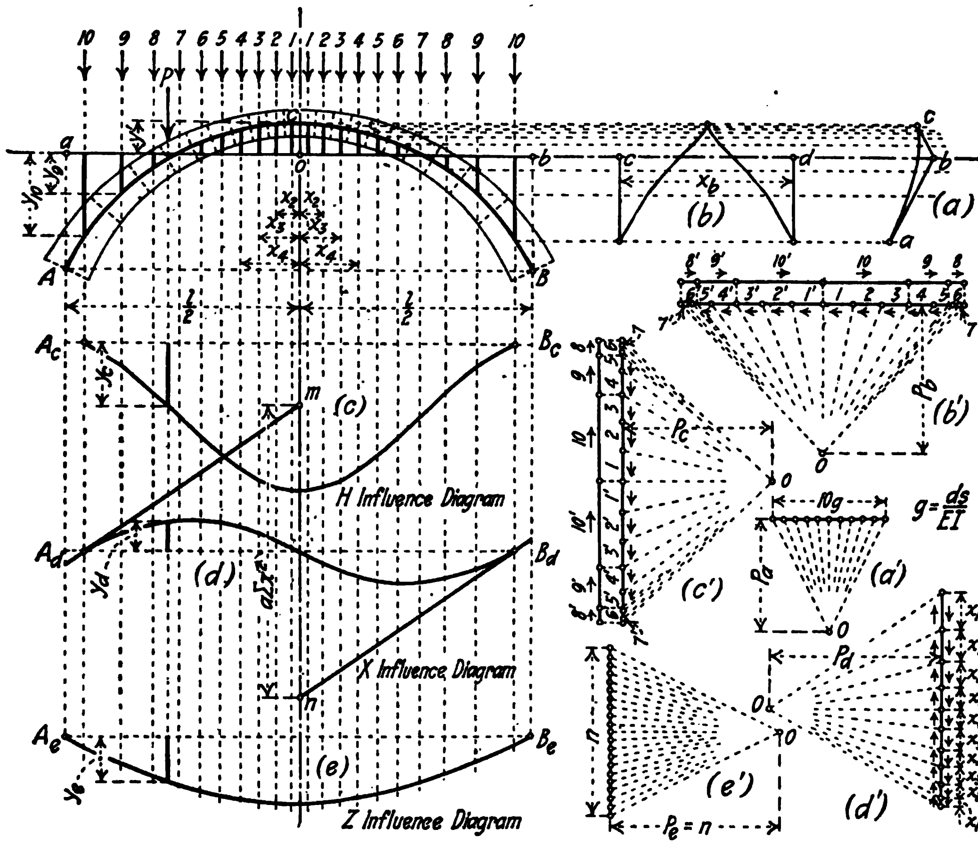



FIG. 11. INFLUENCE DIAGRAMS FOR FIXED ARCH.

**Influence Diagram for H.**—Given an arch with a varying cross-section as in Fig. 11. Divide the arch ring into 20 segments so that $\frac{ds}{E \cdot I} = g =$ a constant. This can be done most easily by means of the graphic method described in Fig. 13. Calculate the gravity axis $a$–$b$. This can be done algebraically, or by graphics by means of force polygon $(a)$, and equilibrium polygon $(a)$.

To calculate $\Sigma y^2$, lay off the values of $y$ as loads in force polygon ($b$), and with an arbitrary pole distance $p_b$, draw the equilibrium polygon ($b$), the strings in equilibrium polygon ($b$), being drawn parallel to the corresponding rays in force polygon ($b$). Then $\Sigma y^2 = x_b \cdot p_b$. To calculate $\Sigma x' \cdot y$ draw equilibrium polygon ($c$) with strings parallel to the rays in force polygon ($c$). Force polygons ($b$) and ($c$) are identical except ($c$) is turned 90° with respect to ($b$). Then the value of $\Sigma x' \cdot y$ at any point will be $= y_c \cdot p_c$, and the horizontal thrust for a load at any point will be

$$H = y_c \cdot p_c / x_b \cdot p_b$$

but $p_b = p_c$, and

$$H = y_c / x_b$$

where $y_c$ is the ordinate in equilibrium polygon ($c$), measured under the load.

The smooth curve tangent to the equilibrium polygon ($d$) is the influence diagram for $H$.

**Influence Diagram for X.**—To calculate values of $\Sigma x'.x$, lay of values of $x$, measured from the center of the arch, as vertical loads in force polygon ($d$), and with an arbitrary pole distance $p_d$, draw equilibrium polygon ($d$), by drawing the strings in equilibrium polygon ($d$), parallel to the rays in force polygon ($d$). Now the value of $\Sigma x' \cdot x$ for a load at any point will be $y_d \cdot p_d$.

Now the value of $y_d$ at center of space $= m-n$, multiplied by $p_d$ will be the value $\Sigma x^2$, and

$$\Sigma x^2 = m-n \cdot p_d$$

and

$$X = \frac{y_d \cdot p_d}{m-n \cdot p_d} = y_d / m-n.$$

The smooth curve tangent to equilibrium polygon ($d$) is the influence diagram for $X$. The value of $X$ for a load, $P$, at any point will be equal to $P \cdot y_d / m-n$.

**Influence Diagram for Z.**—To calculate values of $\Sigma x'$, lay of $n$ equal segments as vertical loads in force polygon ($e$) and with a pole distance $p_e = n$, draw equilibrium polygon ($e$) by drawing strings in equilibrium polygon ($e$) parallel to rays in force polygon ($e$). Then the value of $\Sigma x'$ at any point will be $y_e \cdot n$, and we will have

$$Z = y_e \cdot n / n = y_e.$$

The value of $Z$ for a load $P$ at any point will be

$$Z = P \cdot y_e.$$

The smooth curve drawn parallel to the equilibrium polygon ($e$) will be the influence diagram for $Z$.

The end shears for a unit load may be calculated by equations (39) and (40).

The moments at the ends for a unit load may be calculated from equations (36) and (37).

The moment at any point for a unit load may also be calculated by means of the formula (23),

$$M = M_1 - Hy - X \cdot x - Z \tag{23}$$

Having drawn the influence diagrams the values of horizontal thrust and the moment at the ends and at the crown may be easily calculated for any loads on the arch, and the loading for maximum stresses can be determined.

**Live Loads on Highway Arch Bridges.**—The committee on reinforced concrete highway bridges and culverts, American Concrete Institute, 1914, recommends a 20-ton motor truck or 140 lb. per sq. ft. for class A bridges, and a 15-ton motor truck, and 90 lb. per sq. ft. up to 60 ft. spans for class B bridges. But one motor truck is assumed on the bridge at one time, and the remainder of the bridge is covered with uniform load.

The Iowa Highway Commission requires that arches be designed for a uniform load of 100 lb. per sq. ft. over the roadway and sidewalks, or a 15-ton traction engine so placed as to produce in combination with the dead load stresses maximum stresses in the arch ring.

The Illinois Highway Commission requires that arches be designed for a live load of 125 lb. per sq. ft. or a concentrated load giving a maximum uniform load of 525 lb. per sq. ft., over an area 16 ft. wide and 3 ft. 10 in. long. Sidewalks to have a uniform live load of 125 lb. per sq. ft.

The live loads specified by the author are given in Appendix II.

**Allowable Stresses.**—The allowable stresses in arches are given in the author's "General Specifications for Concrete Bridges and Foundations," in Appendix II.

**Impact.**—Where there is a crown filling of not less than one foot the effect of impact may be neglected. For open-spandrel arches with concrete slab floors the effect of impact should be considered the same as for other concrete bridges.

**Distribution of Loads Through Fill.**—The distribution of live loads when transmitted through filling has been studied but no standard specification has been adopted. A common specification is to assume that the load on a wheel is uniformly distributed over a square the side of which is equal to the width of tire plus twice the depth of the fill.

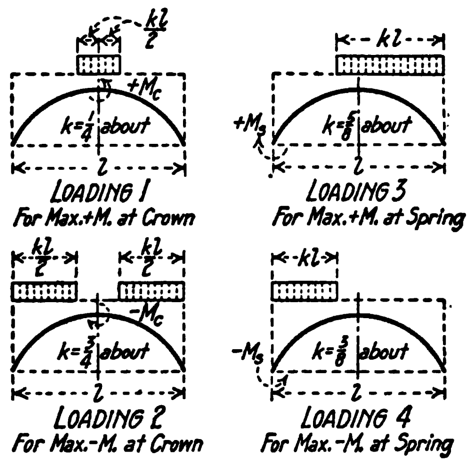



FIG. 12.

The Ohio State Highway Department requires that on masonry arches with spandrel filling three feet or more in depth, the weight of the concentrated load shall be assumed as distributed uniformly over an area 12 ft. wide and 20 ft. long in the direction of the roadway. Where a pavement is used the concentrated load is assumed as distributed over an area whose length is equal to twice the depth of earth fill plus four times the thickness of the pavement.

**Allowance for Temperature.**—Tests made at the Iowa State College of Agriculture and Mechanic Arts and described in Bulletin 30, show that for a latitude of approximately 40 degrees a temperature provision should be made in designing an arch for a variation of 40 degrees F. each way from the temperature of no temperature stress.

The Iowa Highway Commission requires that arches be designed for stresses induced by a temperature range of 80 degrees F.

The Illinois Highway Commission requires that arches be designed for range of 40 degrees F. either way from normal.

Watson's "General Specifications for Concrete Highway Bridges," 1916 edition, requires that arches be designed for a range of 35 degrees each side of normal for a latitude of 40 degrees, and that the limit be increased for higher latitudes and be decreased for lower latitudes.

**Arch Loading.**—Mr. Cochrane* calculated the stresses in arches by means of influence diagrams, and recommends the four typical arrangements of live loads, as shown in Fig. 12. Loadings 1 and 2 are for maximum positive and negative moments at the crown, respectively, and when combined cover the entire span. Loadings 3 and 4 are for maximum positive and negative moments at the springing respectively, and when combined cover the entire span.

If heavy concentrations are specified the method of influence lines should be used in calculating the stresses in arches.

**Division of Arch Ring for $\frac{\Delta s}{E.I}$ = a Constant.**—The arch may be divided in segments in which $\frac{\Delta s}{E \cdot I} = g$ = a constant, by the graphic method shown in Fig. 13. The line $A$–$B$ is made equal to one-half the length of the arch axis. The curve $c$–$g$–$d$ is drawn through points whose ordinates are the values of $I$ and whose abscissas are the corresponding distances along the arch axis from the springing line. A length $A$–$f$ is then assumed for the length of the first segment, and the isosceles triangle $A$–$e$–$f$ is drawn. Starting from point $f$, lines are drawn parallel to $A$–$e$ and

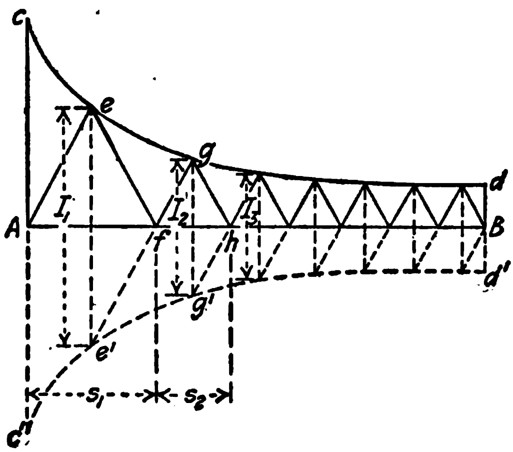



FIG. 13.

$e$–$f$ as shown. If the last division does not check at $B$ the operation must be repeated. The base of each triangle is the length of $\Delta s$ and the altitude is the mean value of $I$, and since all triangles are similar $\Delta s/I$ is a constant. The modification of the method shown by the dotted lines may be used.

**Best Shape of Arch Axis.***—If $l$ = span of arch, $x = c.l$ = distance of any point in arch ring from center line; $y$ = vertical distance of any point in arch ring from tangent to arch ring at center; $r$ = rise of arch; $\phi$ = angle between tangent to arch axis at springing and the horizontal, then the equations that give the best form of arch axis are

For open-spandrel arches,

$$y = \frac{8r \cdot l}{6 + 5r}(3c^2 + 10c^3 \cdot r) \tag{51}$$

$$\tan \phi = \frac{8r}{6 + 5r}(3 + 5r) \tag{52}$$

For filled-spandrel arches

* Design of Symmetrical Hingeless Arches, by Victor H. Cochrane, Proceedings Engineer's Society of Western Pennsylvania, Vol. 32, No. 8.

$$y = \frac{4r \cdot l}{1 + 3r}(c^3 + 24c^5 \cdot r) \tag{53}$$

$$\tan \phi = \frac{4r}{1 + 3r}(1 + 7.5r) \tag{54}$$

It is very common practice to use a form for the arch such that there will be no tensile stresses in the arch for dead load.

**Empirical Rules for Thickness of Arch Ring.***—Joseph P. Schwada gives the following formula for the thickness of highway bridge arches:

$$d = \frac{l^2}{57.6(R - h)f_c \cdot K}\left[\frac{B}{20} + R + 8d + 6F + \frac{w}{20}\right] \tag{55}$$

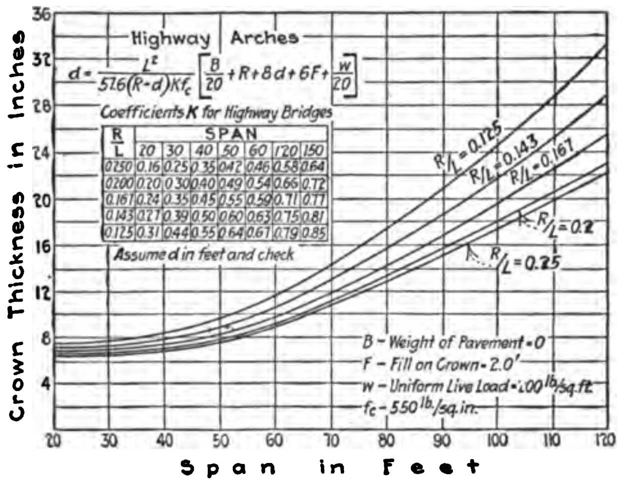



FIG. 14.

where $d$ = crown thickness in feet,
$l$ = clear span in feet,
$R$ = rise of intrados in feet,
$F$ = depth of fill at crown in feet, not including pavement,
$B$ = weight of pavement in lb. per sq. ft.
$w$ = uniform live load in lb. per sq. ft.

The thickness of highway bridge arches for different conditions are given in Fig. 14. In using formula (55) it is necessary to use an approximate value of $d$; if the calculated value and assumed value of $d$ do not check, a new value must be assumed and the thickness recalculated.

**Variation in Thickness of Arch Rib.**†—Mr. Cochrane has made an analysis of the variation in rib thickness to give equal stresses throughout the arch rib. The variation in thickness is shown in Fig. 15. The thickness at the quarter point may be less than at the crown, but an arch of this design would be unsightly and difficult to build. The variation in thickness of arch rib shown by the dotted lines is recommended.

**Reinforcement of Arch Rings.**—Reinforcement in concrete arches makes the action of the structure more certain and permit higher working stresses in the conrcete than can be permitted

* Engineering News, Nov. 9, 1916, p. 880.

† Design of Symmetrical Hingeless Arches, by Victor H. Cochrane, Proceedings Engineer's Society of Western Pennsylvania, Vol. 32, No. 8.

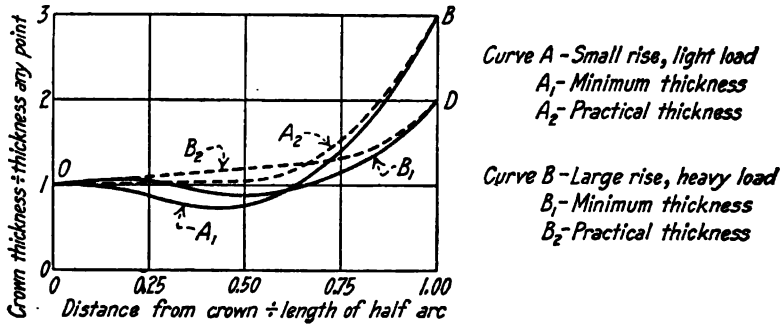



FIG. 15.

on plain concrete arches. Reinforced concrete arches can therefore be built with thinner arch rings and lighter abutments than plain concrete arches. More reinforcement is used than would be required to take the tensile stresses.

It is the best practice to use reinforcement near both surfaces of the arch ring to insure against positive and negative moments. The amount of steel at the crown varies from ½ to 1½ per cent. The author has specified 1 per cent of reinforcement at the crown in "General Specifications for Concrete Bridges and Foundations," in Appendix II. Transverse bars at right angles to the longitudinal bars are generally used to prevent cracks in the concrete and to assist in distributing the loads laterally. Web reinforcement is not ordinarily required for shear, but has the advantage of making the longitudinal and transverse reinforcement act as a unit, and web reinforcement should preferably be used.

For the calculation of stresses due to direct stress and flexure as in arch rings, see Chapter XVIII.

**EXAMPLES.**—The arch bridge shown in Fig. 16 was designed by the Iowa Highway Commission. The bridge was designed for a live load of 100 lb. per sq. ft. or a 15-ton traction engine. The allowable compression in concrete was 650 lb. per sq. in. where no temperature stresses occur, and 750 lb. per sq. in. where temperature stresses are included. The allowable tension in steel was 16,000 lb. per sq. in., with 20,000 lb. per sq. in where temperature stresses were included. The arch rib was designed for a variation of 40 degrees F. from the mean. The arch ring, abutments and spandrels were built of 1–2–4 mix Portland cement concrete.

The arch bridge shown in Fig. 17 was designed by the Michigan State Highway Department. The bridge was designed for a uniform live load of 100 lb. per sq. ft. or an 18-ton road roller. The arch ring was built of 1–2–4 mix Portland cement concrete, while the abutments were built of 1–3–6 mix Portland cement concrete. The concrete in the arch rib was designed for a compression of 650 lb. per sq. in. The allowable tension in steel was 16,000 lb. per sq. in.

**Rainbow Arch Bridge.**—The arch bridge in Fig. 18, built at Carmi, Ill. in 1916, has three arch spans of 90 ft. each between piers, with a rise of 18 ft. and a radius of 65 ft. 3 in. on the under side. It has an 18-ft. roadway. The arch was constructed by placing the structural steel reinforcing of the ribs first, connecting them rigidly in place by struts, floorbeams and hangers. The formwork was then built around the steel reinforcing. The reinforcing for the arch ribs is composed of four angles laced on four sides and with the backs of the angles turned outward. The concrete was 1:2:4 mix using gravel with a maximum size of 1½ in.

The bridge was designed for a live load of 125 lb. per sq. ft. and a 20-ton road roller. The stresses were those required by the specifications of the Illinois Highway Commission. The plans were prepared by the Marsh Bridge Company, which has patented certain features of the bridge.

The cost of the bridge in 1916 was $21,960.

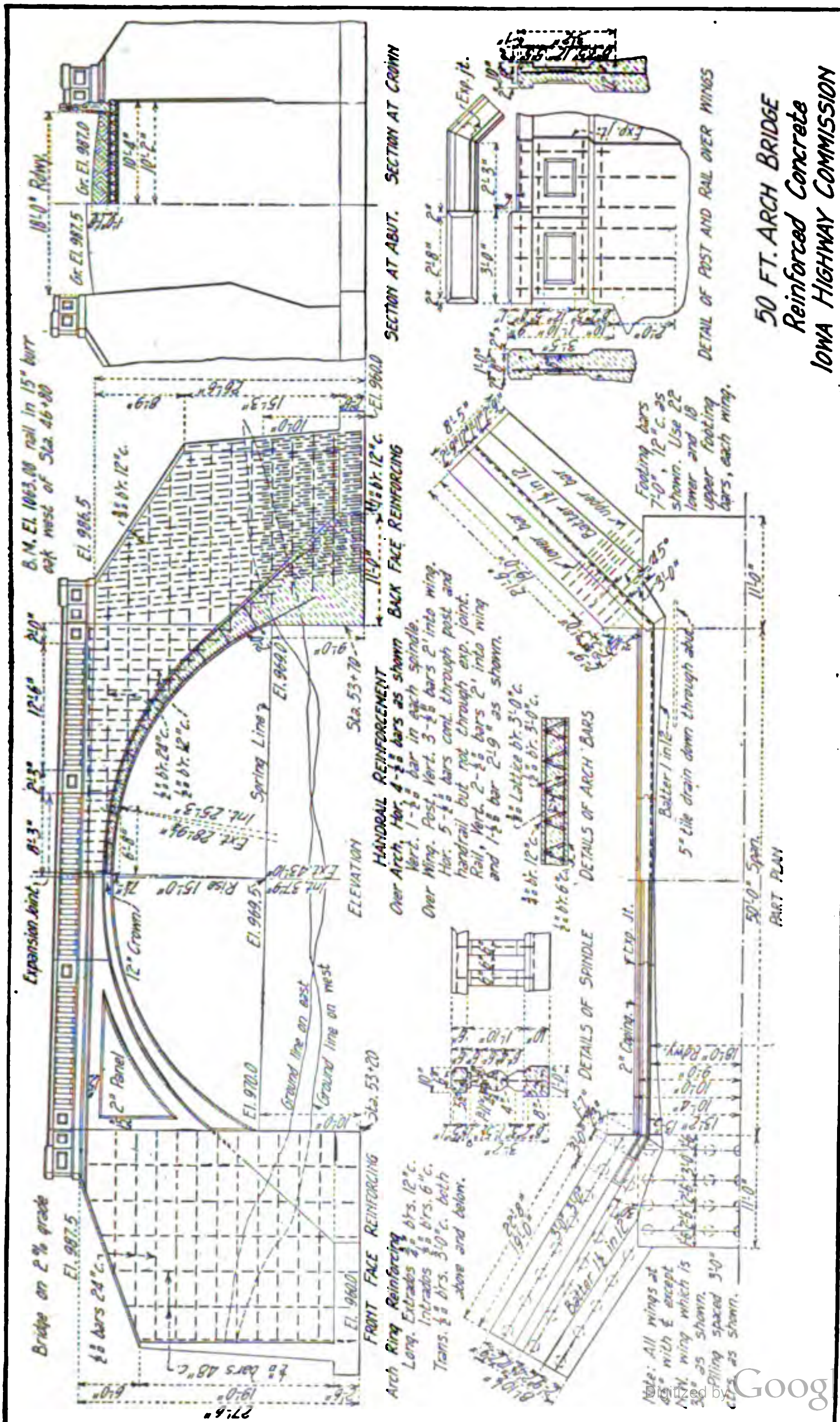

50 FT. ARCH BRIDGE
Reinforced Concrete
IOWA HIGHWAY COMMISSION
SECTION AT ABUT.
SECTION AT CROWN
FRONT FACE REINFORCING
BACK FACE REINFORCING
ELEVATION
HANDRAIL REINFORCEMENT
DETAILS OF SPINDLE
DETAILS OF ARCH BARS
DETAIL OF POST AND RAIL OVER WINGS
PART PLAN
Expansion Joint
Spring Line
Bridge on 2% grade
50'-0" Span

FIG. 16.

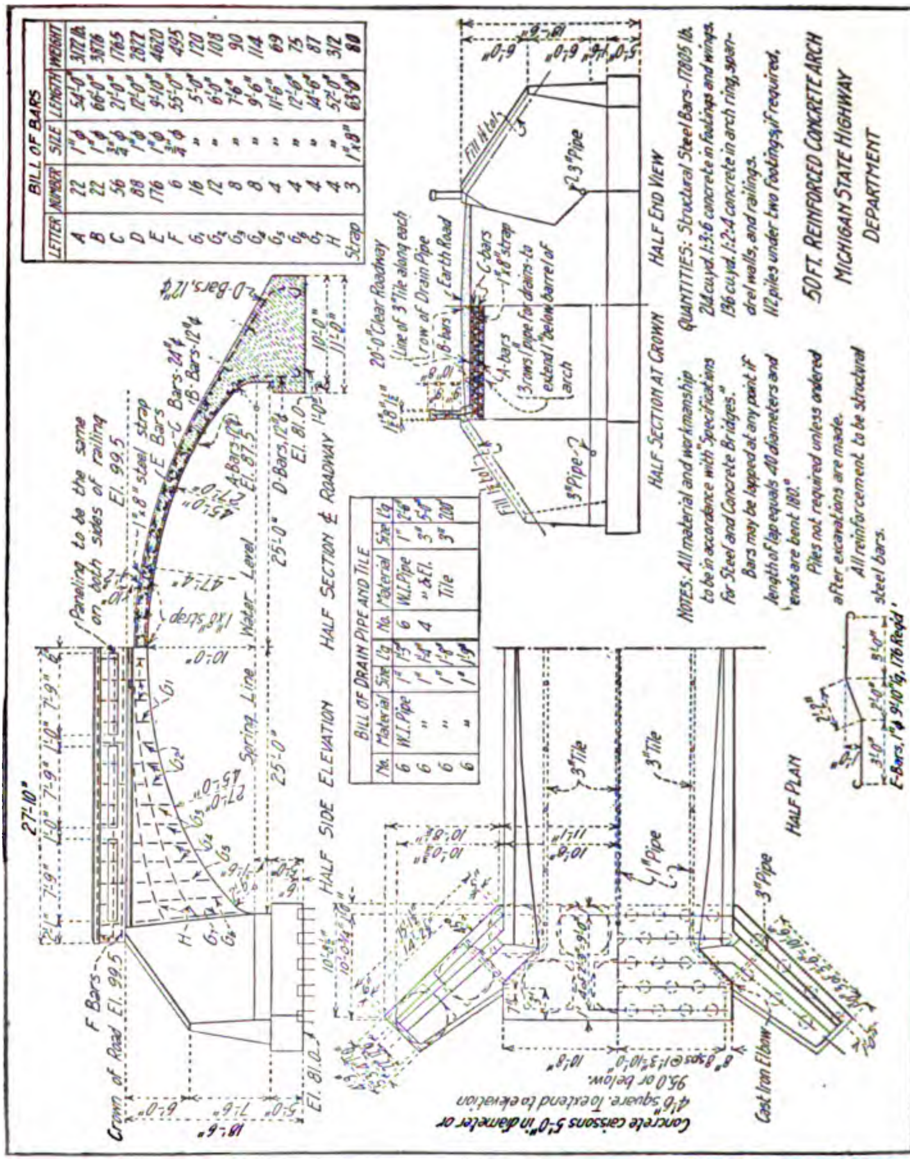



FIG. 17.

**REFERENCES.**—For excellent algebraic solutions, see Turneaure and Maurer's "Principles of Reinforced Concrete Construction," Taylor and Thompson's "Concrete, Plain and Reinforced," and Hool and Johnson's "Concrete Engineer's Handbook."

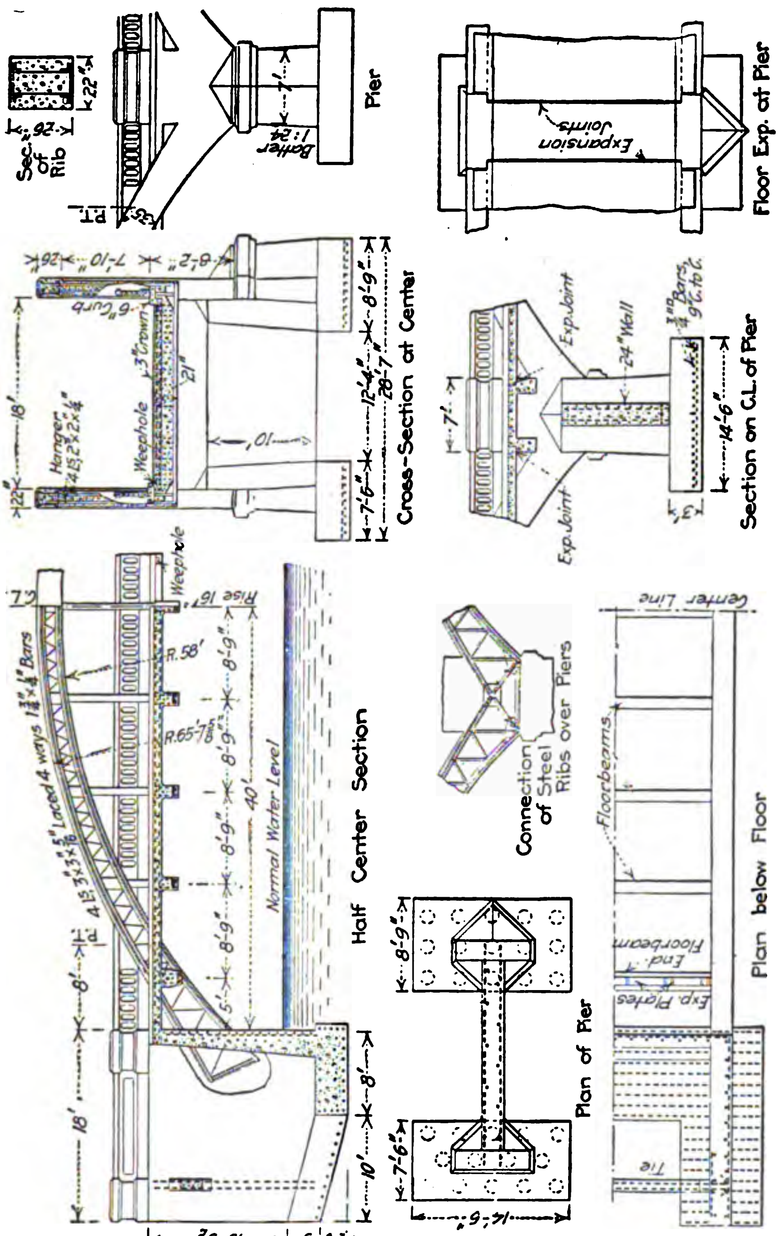



FIG. 18. RAINBOW ARCH BRIDGE, CARMI, ILL.

# PART IV.

# CONSTRUCTION OF HIGHWAY BRIDGES.

## CHAPTER XXIV.

### BRIDGE ENGINEERING.

**Introduction.**—The discussion in this chapter will include Bridge Surveys, Bridge Plans, Bridge Contracts, and Specifications.

For a detailed discussion of Structural Drafting, see Chapter XII of the author's "Structural Engineers' Handbook."

**BRIDGE SURVEYS.**—Before beginning the plans for a highway bridge, a very careful and complete survey should be made of the proposed bridge site. The information should be complete enough to enable the designer to decide upon the proper type and details of the structure which is best suited to the conditions. The survey should include the following data.

1. Location of bridge together with distance from shipping points.

2. Complete data with careful sketch plans of the present structure, and substructure.

3. Data on high and low water together with a report on the efficiency of the present waterway.

4. Survey of proposed bridge site which should include

*a.* Bench mark.

*b.* Survey should be tied to section corners or definite and known land lines.

*c.* The alignment of the road should be given on each side of the bridge. The skew of the center line of the bridge should be given with the channel.

*d.* The plan should show the course of the stream, the bank lines, old channels, dykes or spoil banks, direction of flow during flood stage, bank erosion, etc.

The plan should show the location of the old structure and a proposed location of the new structure.

The plan should show contours of the stream together with high and low water marks.

Soundings should be made to determine the character of the foundation material. The soundings may be made by boring, by driving pipes or by digging pits. For an important bridge crossing two sets of borings should be made. (1) Borings should be made along the center line, and from them if favorable the location of the piers and abutments should be determined. (2) After tentative locations of the piers additional borings should be made on the site of each pier or abutment, at least one at each corner, and intermediate ones if necessary. The data should include the lowest water level and depth of foundations, character of material upon which the foundations will rest, data on number, spacing and length of piles if they are to be used. An estimate should be made of the safe load on the foundations, and a recommendation should be made as to the best type of foundation. The liability of the bank to scour should be noted, and suggestions should be given as to the best means, such as riprap or sheet piling to prevent it.

The plan should show the location of bench marks, alignment hubs and reference points, section and township numbers, width of right of way, property owners of adjacent property, directions, scales, and other data.

*e.* The profile should show the present and proposed road grades; the cross-sections of the channel on center line, and if the channel is irregular cross-sections on lines parallel to the center

line. The high and low water marks should be shown and the date of high water should be noted. All runoff data should be recorded. Soundings should be made of the stream and data should be obtained on scour. All overflow openings should be recorded. All intersecting road grades should be recorded.

*f.* A survey should be made of the stream for a distance varying from 500 ft. to 1,000 ft. on both sides of the bridge site to determine the fall, the cross-section and hydraulic properties of the stream bed that will make it possible to make an estimate of the flow of the stream, and also the necessary clear channel at the bridge site. The clear openings of other bridges on the same stream should be noted together with notes on the high water marks, and data on the adequacy of the waterway opening.

*g.* Surveys should be made on the center line of the highway with cross-sections at intervals of 100 ft. or less, so that the cost of constructing approaches may be estimated.

*h.* If the stream has a crooked channel a survey should be made to obtain complete data to determine whether or not a channel change is necessary.

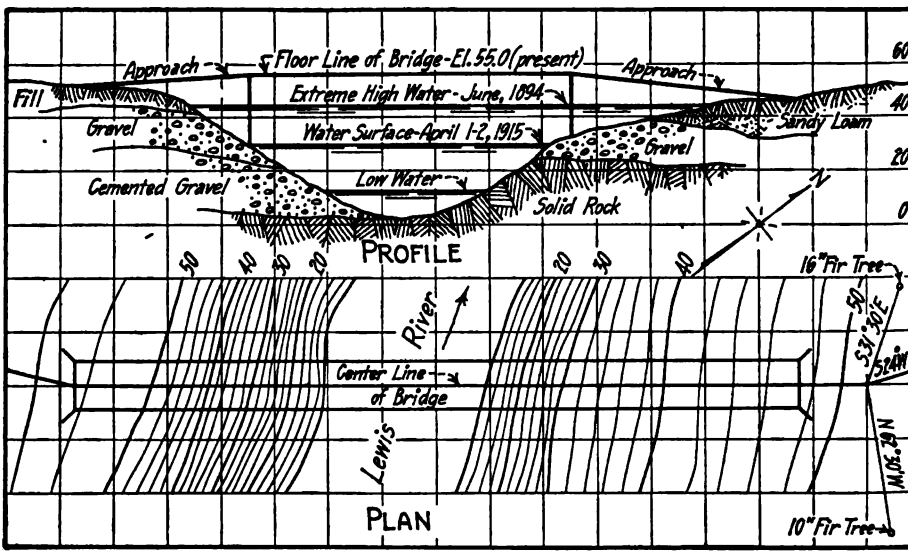



FIG. 1. PROFILE AND MAP OF BRIDGE SITE. OREGON HIGHWAY COMMISSION.

*i.* Recommendations should be made by the engineer in charge of the survey as to (1) type of structure; (2) number and length of spans; (3) width of roadway; (4) proposed grade elevation; (5) special loading requirements; (6) depth of footings and type of foundations; (7) special features of design, such as light poles, sidewalks, name plate, retaining walls.

A plan and a profile of a bridge site as prepared by the Oregon Highway Commission is given in Fig. 1.

The following data are required to be furnished by the Oregon Highway Commission.

## SURVEY OF SITE.

GENERAL INFORMATION (Data Required.)

(Please answer every question)

1. Width of roadway desired . . . . . . . . . . . . . . . . . . . . . . . .
2. Type of floor wanted—wood or concrete . . . . . . . . . . . . . . . . . . . . . .
3. Type of joists wanted—wood or steel . . . . . . . . . . . . . . . . . . . . . . . .

4. Number of sidewalks required ........................................
5. Width of sidewalks ........................................
6. Type of bridge preferred—steel, concrete, or wood ........................................
7. Give date and elevation of extreme high water ........................................
8. Give date and elevation of average low water ........................................
9. Is the crossing on a "trunk" or secondary road ........................................
10. If grade has been established show elevations on profile ........................................
11. Distance from bridge-site to nearest railroad station ........................................
12. Condition of road from railroad station to bridge-site ........................................
13. Can sand and stone for concrete be obtained locally. If so, give distance from bridge-site..
14. Can timber for falsework and piling be obtained locally. If so, give distance from bridge-site.
15. Is there a bridge at this site at present. If so, indicate location on map and profile.........
16. Will it be necessary to maintain traffic during construction of new bridge................
17. Is design of approaches desired ........................................
18. If earth-filled approaches prove economical, is suitable material available for filling.........
19. Will the approaches be constructed by county forces ........................................
20. Give date of desired completion of structure ........................................
21. Give direction in which roads will approach bridge ........................................
22. Note if there will be plenty of clearance for swinging teams in narrow canyons or bluff surroundings ........................................

## MAP AND PROFILE OF BRIDGE-SITE.

Over ........................................
(Creek or River)
in Section......................., Township..............., Range..............M.,
near ........................................, ........................................
(Town or City) (County)

**BRIDGE PLANS.**—The plans for a bridge must contain all the information necessary for the design of the structure, for ordering the material, for fabricating the bridge in the shop, for erecting the structure, and for making a complete estimate of the material used in the structure. Every complete set of plans for a bridge must contain the following information, in so far as the different items apply to the particular structure.

1. **General Plan.**—This will include a profile of the ground; location of the structure; elevations of ruling points in the structure; clearances; grades; direction of flow, high water, and low water; and all other data necessary for designing the substructure and superstructure.

2. **Stress Diagram.**—This will give the main dimensions of the bridge, the loading, stresses in all members for the dead loads, live loads, wind loads, etc., itemized separately; the total maximum stresses and minimum stresses; sizes of members; typical sections of all built members showing arrangement of material, and all information necessary for the detailing of the various parts of the structure.

3. **Shop Drawings.**—Shop detail drawings should be made for all steel and iron work and detail drawings of all timber, masonry and concrete work.

4. **Foundation or Masonry Plan.**—The foundation or masonry plan should contain detail drawings of all foundations, walls, piers, etc., that support the structure. The plans should show the loads on the foundations; the depths of footings; the spacing of piles where used; the proportions for the concrete; the quality of masonry and mortar; the allowable bearing on the soil; and all data necessary for accurately locating and constructing the foundations.

5. **Erection Diagram.**—The erection diagram should show the relative location of every part of the structure; shipping marks for the various members; all main dimensions; number of pieces in a member; packing of pins; size and grip of pins, and any special feature or information that

may assist the erector in the field. The approximate weight of heavy pieces will materially assist he erector in designing his falsework and derricks.

6. **Falsework Plans.**—For ordinary structures it is not common to prepare falsework plans in the office, this important detail being left to the erector in the field. Erection plans should be worked out in the office, and should show in detail all members and connections of the falsework, and also give instructions for the successive steps in carrying out the work. Falsework plans are especially important for concrete and masonry arches and other concrete structures, and for forms for all walls, piers, etc. Detail plans of travelers, derricks, etc., should also be furnished the erector.

7. **Bills of Material.**—Complete bills of material showing the different parts of the structure with its mark, and the shipping weight should be prepared. This is necessary in checking up the material to see that it has all been shipped or received, and to check the shipping weight.

8. **Rivet List.**—The rivet list should show the dimensions and number of all field rivets, field bolts, spikes, etc., used in the erection of the structure.

9. **List of Drawings.**—A list should be made showing the contents of all drawings belonging to the structure.

**DESIGN PLANS.**—The preliminary plans of steel bridges may consist (1) of a stress diagram showing the stresses, dimensions and sizes of the principal members of the bridge, and also standard specifications as given in Appendix I; (2) of detail plans which show the make-up of all the members together with the maximum and minimum spacing of the rivets, thickness and sizes of plates, lacing bars, etc., and also standard specifications; and (3) of completely detailed shop plans and specifications. When properly carried out all of the methods will give satisfactory results. Ordinarily the customer cannot understand the details of the bridge from a study of the stress diagram and finds the second and third methods much more satisfactory. It is seldom profitable to prepare shop plans until after the order for the bridge has been placed in the shop, and requisitions have been made for the material. On the whole, the method of preparing the preliminary plans, as described in (2), is the most satisfactory. This makes it possible to specify exactly the details of the sections and at the same time permits the bridge shop to follow its own methods wherever possible. The shop practice in different shops differs so much that it is ordinarily cheaper for the bridge company to prepare its own shop plans than to follow shop plans that have been prepared by engineers that are not familiar with the particular shop.

The plans for concrete bridges should be prepared as shown in Chapters XIX to XXIII, inclusive, and specifications as in Appendix II.

**BRIDGE CONTRACTS.**—The contracts for building highway bridges are ordinarily let by county commissioners, county surveyors or other county officers. In a few states contracts for building bridges are let by state or highway engineers. The common method of awarding contracts for highway bridges has been about as follows: Three or four weeks before the date set for the bridge letting the county clerk or other officer advertises that bids will be received up to a certain hour for building a certain bridge or bridges, and that the bids will then be publicly opened and the contract awarded. The main dimensions and the capacity only are ordinarily specified and the bidders are asked to submit their own plans and specifications. When the various bids and plans are received the commissioners are entirely at a loss as to what is the best thing to do, and the result is that either the contract is given to the lowest bidder on a very poor plan or is given to a favorite bidder on a plan that results in a worse bridge. This loose method of contracting for bridges makes it practically impossible for even honest officials to procure a satisfactory structure and opens up the way for dishonest officials and contractors to arrange a deal whereby the public comes out second best. It also makes it possible for the contractors to "pool" so that the bridge contract will go to a member of the pool at an agreed price. The county surveyor or local engineer is ordinarily not much better posted on the merits of the bids and

plans than the commissioners, and his participation in the letting does not ordinarily improve matters.

The practice of "bridge pooling" is disreputable and has worked to the disadvantage of both the public and of reputable bridge companies. It has made it possible for "fake bridge companies" to exist and also for crooked public officials to receive part of the profits of the transaction. It has uniformly resulted in high prices and poor bridges.

Before advertising for bids the matter of the design of the bridge or bridges should be placed in the hands of a competent consulting bridge engineer. Detail plans and specifications should be prepared and an estimate of the probable cost submitted to the officials. All bids should then be received on the official plans and specifications. If the bids are too high they should all be rejected and the work readvertised on the same or on revised plans. The bridge contractor takes a considerable risk and is entitled to a good legitimate profit, and the engineer should add 15 to 20 per cent for profit to his estimated cost. No work should be done at the shop until after the shop plans have been checked and approved by the consulting engineer. The shop, field and final inspection should be in the hands of the consulting engineer. This method will meet the approval of all legitimate bridge companies, and will result in better bridges at a less cost.

In several states the construction of highway bridges is supervised by the state highway commission, while in other states the state highway commission gives assistance in preparing plans and constructing highway bridges when requested so to do.

**ADVERTISEMENT FOR BIDS.**—To obtain bids from responsible bidders the bridge letting should be advertised in the local papers and in the technical press.

The following advertisement will serve to show what data should be furnished prospective bidders.

BRIDGE ADVERTISEMENT.

BRIDGES AND CULVERTS.

KEWANEE, ILLINOIS.

Sealed proposals will be received by the Board of Commissioners of Road Improvement District No. 1, of Henry County, Illinois, at Kewanee, Illinois, until eleven o'clock A.M., Aug. 7, 1919, for the construction of bridges and culverts as follows:

| | |
|---|---|
| 50-ft. Steel Bridge | 24 ft. wide |
| 50-ft. Steel Bridge | 20 ft. wide |
| 152-ft. Concrete Viaduct | 24 ft. wide |
| 160-ft. Concrete Viaduct | 20 ft. wide |
| 4 Concrete Culverts | 4 ft. by 5 ft. |

Proposals must be submitted on the form furnished by the Board and must be accompanied by a certified check, payable to the Treasurer of Road Improvement District No. 1, Henry County, Illinois, for not less than five per cent of the total amount of the bid.

Plans, specifications and estimate of quantities may be examined at the office of the Consulting Engineer, Kewanee, Ill., and at the office of the Board of Commissioners of Road Improvement District No. 1, Kewanee, Illinois.

Plans will be furnished by the undersigned upon receipt of deposit of five ($5) dollars which will be refunded upon safe return of plans.

The right is reserved to reject any or all bids and to waive any informality in the bids received.

BOARD OF COMMISSIONERS OF ROAD IMPROVEMENT
DISTRICT NO. 1, HENRY COUNTY, ILLINOIS.
JOHN C. JONES, Consulting Engineer.

**CONTRACT.**—After the contract has been awarded the contract papers should be drawn up and signed, and an indemnity bond should be furnished by a good surety company. Sample contract and bond forms used by the author follow:

## BRIDGE CONTRACT.

**Agreement** made this .............. day of ................, 19...., by and between ........................, a corporation of the State of ................, party of the first part, and ........................, party of the second part.

**Witnesseth,** that for the consideration and upon the terms and conditions hereinafter provided, the party of the first part agrees to furnish all material and labor therefor, and construct and erect in a good and workmanlike manner ....................................over the .................... called ................ at a point where the .................... crosses said ................ in the ................ of ...................., County of ................, and State of ................, according to the attached plans and specifications which are made a part of this contract: The bridge is to have ........ spans; extreme length of each span, ............; space between the face of abutments, .................; roadway, ........ feet clear; sidewalk, ........ feet clear. The abutments to be ........ .................; the piers to be ....................

It is further agreed that the said first party shall save and hold said second party free and harmless from any and all claims for damages to life, limb or property occasioned or caused by said first party's employees; and from all claims for materials and labor furnished on this contract.

The party of the first part agrees to complete the work herein contracted for, and to have the said bridge open and ready for travel on or before the ............ day of .............., 19.....

The party of the second part agrees to pay the party of the first part for said bridge the sum of ........................ in cash, as follows: ........ per cent upon the delivery by the party of the first part of the steel and other material on the bridge site for said bridge, ........ per cent additional upon the completion of the erection of the different parts of the bridge, and the remaining ........ per cent upon the completion and acceptance of the bridge by the consulting engineer of the second party. Estimates of material delivered and work done shall be made by the consulting engineer not later than the 5th day of the month for all material delivered or work done during the preceding month, and the payment will be made on or before the 15th of the month for the material delivered and work done the preceding month.

It is further stipulated and agreed that the party of the first part shall furnish to the party of the second part an indemnity bond in an approved surety company in the sum of ............ ........ dollars.

It is further stipulated and agreed that for the failure of the first party to complete the bridge as stipulated the said first party shall forfeit ............ dollars for each working day until the bridge is completed. This sum to be considered as liquidated damages, and not as a penalty.

(It is further stipulated and agreed that the party of the first part shall not be held responsible for delays in delivery of material from the rolling mills, nor delays in transportation, nor for delays occasioned by strikes, fires, floods, storms, or other circumstances beyond its control, but may be granted an extension of time as may be determined by the consulting engineer of the second party.)

**In Witness Whereof,** the said parties to this agreement hereunto set their hands and seals as of the day and year first above written.

......................First Party.
......................Second Party.

## BOND.

**Know all Men by These Presents,** that .........................., as party of the first part, and the ......................., a corporation organized and existing under the laws of the State of ................ as Surety, are held and firmly bound unto the .......... .........., party of the second part, its successors and assigns, in the sum of ................ dollars, lawful money of the United States, to the payment of which sum well and truly to be

made the said first party and the said Surety do hereby bind themselves, their heirs, executors, administrators, successors and assigns, jointly and severally, firmly by these presents.

Signed, Sealed, Dated and Delivered, this ............ day of ................, 19.....

WHEREAS, the said party of the first part has entered into a certain contract with the ........................, dated ...................., 19...., for ......................
................................................................................
................................................................................
which contract is hereto attached and made a part hereof, and for a fuller description thereof reference is made to said contract:

NOW, THEREFORE, THE CONDITION OF THIS OBLIGATION IS SUCH, that if the said party of the first part shall and does pay as they become due, all just claims for all work and labor performed and all skill and material furnished in the execution of such contract, and, also, shall save the party of the second part named in this bond harmless from any cost, charge and expense that may accrue on account of the doing of the work specified in such contract according to the terms thereof and the contract price therein, and shall comply with all the requirements of the law, then this obligation shall be void; otherwise to remain in full force and effect.

IN TESTIMONY WHEREOF, We have hereunto set our hands and seals, this .......... day of ......................., 19.....

Signed, sealed and delivered
in the presence of

.........................

......................... ............................Seal.

............................Seal.

STATE OF.................... }
COUNTY OF.................. } ss.

On this .......... day of .................., 19...., before me, a Notary Public in and for the County and State aforesaid, personally came ......................................
................................................................................
to me known, who, being by me duly sworn, did depose and say that he resided in ............ ................; that he is the ...................... of the ....................... ........................., the corporation described in and which executed the above bond as party of the first part; that he knew the seal of said corporation; that the seal affixed to said instrument was such corporate seal; that it was so affixed by order of the board of directors of said corporation and that he signed his name thereto by like order.

.......................................
Notary Public.

STATE OF................ }
COUNTY OF.............. } ss.

On this ............ day of ...................., 19...., before me, a Notary Public in and for the County and State aforesaid, personally came ............................ to me known, who, being by me duly sworn, did depose and say that he resided in ............ ....................; that he is the .................... of the ....................... ...................., the corporation described in and which executed the above bond as Surety; that he knew the seal of said corporation; that the seal affixed to said instrument was such corporate seal; that it was so affixed by order of the board of directors of said corporation and that he signed his name thereto by like order.

.......................................
Notary Public.

## GENERAL SPECIFICATIONS FOR CONSTRUCTION OF A HIGHWAY BRIDGE.*

## GENERAL CLAUSES.

1. **Intent of Specifications.**—The intent of these specifications is to provide for the erection and completion of the work described herein in detail, and it is understood that the contractor for all or any part will furnish all labor, materials, tools, transportation and necessary supplies to make each part complete. Any deviation from these requirements must be stipulated in the contract.

A profile and bridge plan will be furnished, but each bidder must satisfy himself by examinations as to the character of the material and all local conditions affecting the contract.

2. **Working Drawings.**—The contractor shall prepare such working or shop plans as may be required to properly detail the work. The shop and working drawings shall be submitted to the engineer for his approval. All materials ordered or work done by the contractor before the shop and working drawings have been approved by the engineer shall be done at the risk of the contractor. Three copies of shop and detail drawings shall be furnished to the engineer for checking and approval; additional copies may be required by the engineer.

3. **Proposals.**—The right is reserved to reject any and all bids, and to waive any informality. All proposal guarantees will be returned immediately after opening the bids, except those of the three lowest bidders, which will be returned within three days after the execution of the contract, or rejection of all bids.

4. **Estimates of Quantities.**—Preliminary estimates of quantities furnished by the engineer are intended only as a check on the contractor's figures and are not guaranteed to be accurate. The estimate of quantities for final payment will be prepared by the engineer as the structure is constructed.

5. **Changes in Plans.**—The plans are prepared for the known conditions. Increase or decrease in quantities necessary to provide for actual conditions will not impair this contract. If any changes are made in the plans, the price to be paid for such changes must be agreed upon in writing at the time such changes are ordered, and before the work is done by the contractor.

6. **Liability Insurance.**—The contractor shall carry liability insurance to protect the public from injuries sustained by reason of carrying on this work, to protect the workmen employed on the work, or to meet the requirements of the workmen's compensation law, if one be in force.

7. **Patented Construction.**—The purchaser assumes all responsibility for defending all suits for infringement of any patent infringed or claimed to be infringed by the design or type of any structure provided for under plans furnished to the contractor, and to pay any royalty which said patentee is entitled to under judgment of the court, if any, and to hold said contractor harmless on account of such suits for royalty.

The contractor assumes all responsibility for defending all suits brought for the infringement of any patent claimed to be infringed by any plans submitted by him, or by any process which he may use in the erection, construction or completion of the structure, provided for in any plans furnished by the purchaser, or by the contractor, and to hold the purchaser harmless on account of any suits or claims for royalty.

8. **Supervision.**—The work of construction is to be carried on in strict conformity with the plans, specifications and drawings prepared, and such additional instructions as may be given from time to time by the engineer. All materials and workmanship shall at all times be subject to inspection by the engineer or his deputy.

The engineer shall be furnished with every reasonable facility for ascertaining that the work is in accordance with the requirements and intention of the contract and specifications, even to the extent of uncovering or taking down portions of finished work. Should the work thus exposed or examined prove satisfactory, the replacing or making good of the parts removed shall be paid for at the contract price for the class of work done, but should the work removed or exposed prove unsatisfactory, the work shall be replaced with satisfactory material at the expense of the contractor.

9. **Defective Work.**—The inspection of the work shall not relieve the contractor of any of his obligations to fulfill his contract, and defective work shall be made good, and unsuitable materials may be rejected, notwithstanding that such work and materials have been previously overlooked by the engineer and accepted or estimated for payment. Any work found defective shall be made good by the contractor in a manner satisfactory to the engineer, and all material determined by the engineer as unsuitable or not in conformity with the specifications shall forthwith be removed by the contractor from the site.

* To accompany "The General Specifications for Steel Highway Bridges" in Appendix I, and "The General Specifications for Concrete Highway Bridges and Foundations" in Appendix II.

10. **Claims for Delays.**—The contractor shall not be entitled to any claim for damages for any hindrance or delay from any cause whatever in the progress of the work or any portion thereof, but such hindrance may entitle him to an extension of time for completing same sufficient to compensate for the detention, provided the purchaser shall have immediate notice from the contractor in writing of the cause and length of detention at the time of its occurrence. The acceptance of any part of the work subsequent to the said date shall not be deemed a waiver of the purchaser to abrogate the contract for delay.

11. **Contractor's Plant and Methods.**—The contractor shall provide and install such a construction plant, and shall use such methods and appliances for the performance of the work to be done under these specifications as will secure a satisfactory quality of work, and a rate of progress which in the opinion of the engineer will insure completion of the work within the time specified. If, at any time before commencing or during the progress of the work, such methods or appliances appear to the engineer to be unsafe, inefficient or inadequate for securing the safety of the workmen, the quality of the work or the rate of progress required, he may order the contractor to increase their safety and efficiency, or to improve their character, and the contractor shall comply with such orders; but the failure of the engineer to make such demand shall not relieve the contractor from his obligations to secure the safe conduct, the quality of the work and the rate of progress required by these specifications, and the contractor alone shall be responsible for the safety, efficiency and adequacy of his plant, appliances and methods.

12. **Damages for Failure to Complete on Time.**—The failure to complete the work covered by these specifications before . . ., will work serious injury to the purchaser. On account of the difficulty of determining the damages arising out of the failure to complete this contract on time the sum of $................ per day for each working day is hereby agreed upon, not as a penalty, but as liquidated damages which the purchaser will suffer by reason of such default. The purchaser shall have the right to deduct the amount of any such damages from any money due or to become due to the contractor under this contract.

13. **Liens.**—If at any time before or within 30 days after this work has been completed any person or persons claiming to have performed any labor or furnished any materials used in the completion of this work shall file with the purchaser any such notice as is described in the lien law, the purchaser shall retain until the discharge thereof, from the money under its control, so much of said moneys as shall be sufficient to satisfy and discharge the amount in such notice claimed to be due, together with the costs of any action brought to enforce such lien created by the filing of such notice.

14. **Contractor's Office.**—The contractor shall maintain an office at the site of the work, at which he or his authorized agent shall be present at all times while the work is in progress. Instructions from the engineer left at this office shall be considered as delivered to the contractor. Copies of the contract, the working drawings and the specifications for the work shall be kept at said office, ready for use at any time. The contractor shall furnish accommodations for the engineer's inspectors in this office.

15. **Engineer's Orders.**—Whenever the contractor is not present on any part of the work, directions or orders may be given by the engineer, and shall be received and obeyed by the superintendent or foreman who may have charge of the particular part of the work in reference to which the orders are given.

16. **Laws and Regulations.**—The contractor shall comply with all laws and municipal ordinances and regulations in any manner affecting those engaged or employed in the work or the materials used in the work or in any way affecting the conduct of the work.

17. **Payment of Laborers.**—The contractor shall punctually pay the workmen who shall be employed on the work covered by this contract, in cash or its equivalent.

18. **Assignment.**—The contractor shall give his personal attention constantly to the faithful prosecution of the work, and shall be present, either in person or by a duly authorized representative, on the site of the work, continually during its progress, to receive instructions or directions from the engineer; he shall not assign, transfer, convey, sublet or otherwise dispose of this contract, or his right, title or interest in or to the same or any part thereof without the previous consent in writing of the purchaser and of the engineer; and he shall not assign by power of attorney or otherwise any of the money to become due and payable under this contract, unless by and with the like consent signified in like manner. If the contractor shall, without previous written consent, assign, transfer, convey, sublet or otherwise dispose of this contract or of his right, title or interest therein, or any of the moneys to become due under this contract to any other person, company or other corporation, this contract may at the option of the purchaser be revoked and annulled, and the purchaser shall be relieved and discharged from any and all liability and all obligations growing out of the same.

19. **Responsibility of Contractor.**—The contractor shall take all responsibility of the work, shall bear all losses resulting to him on account of the amount or character of the work, or because the nature of the work to be done is different from that which he assumes or expects, or on account of the weather, floods or other causes; and he shall assume the defense of and indemnify and save harmless the purchaser and the engineer, their officers and agents from all claims arising from the performance of this contract.

20. **Liability for Accidents.**—The contractor shall, during the performance of the work, take all necessary precautions for the prevention of accidents and shall indemnify and save harmless the purchaser, the engineer, their officers and agents from all damages and costs to which they may be put by reason of injury to persons or property.

21. **Abandoment of Work.**—If the work to be done under this contract shall be abandoned by the contractor, or if the work shall be assigned, or if at any time the engineer shall be of the opinion and shall so certify in writing to the purchaser that the performance of the contract is unnecessarily or unreasonably delayed, or that the contractor is wilfully violating any of the conditions or covenants of the contract or of the specifications, or is executing the same in bad faith or not in accordance with the terms thereof, or if the work be not fully completed within the time mentioned in this contract for its completion, the purchaser shall notify the contractor to discontinue the work or such part thereof, and the purchaser shall, after three days, have the power to contract for the completion of the work, or to place such persons as he may deem advisable by contract or otherwise to complete the work herein described or such part thereof, to take possession of and use any of the materials, plant, tools, equipment, supplies and property of every kind provided by the contractor for the purposes of this work, and to procure other materials for the completion of the same and charge the expense of such labor and materials to the contractor. The expense so charged shall be deducted and paid by the purchaser out of such moneys as may be due or may at any time thereafter become due to the contractor under and by virtue of this work or any part thereof. And in case such expense shall exceed the amount which would have been payable under the contract if the same has been completed by the contractor, he shall pay the amount of such excess to the purchaser, and in case such expense shall be less than the amount which would have been payable under this contract if the same had been completed by the contractor, he shall receive the difference.

22. **Payments.**—If stipulated in the contract, the contractor will receive monthly estimates based on the materials furnished at the site and work completed that may be acceptable to the engineer. All monthly estimates shall be approximate only, and shall be subject to the final estimate. Unless otherwise stipulated in the contract payments of 85 per cent of the monthly estimates will be paid on or before the tenth day of the month following the one in which the material is delivered or the work constructed.

23. **Extra Work.**—Materials or labor required to be furnished for the completion of the work covered by these specifications and not included in the contract will be paid for on the basis of the actual cost of materials furnished and labor performed plus ten per cent (10%) to cover superintendence and profit. No extra work is to be done except on written order of the engineer, whose decision is to be final in the settlement of any dispute that may arise as to the cost of such extra work furnished. All bills for extra work shall be submitted in detail to the engineer for his approval within 30 days after the furnishing of such materials or labor for which extra payment is claimed.

24. **Completion of Contract.**—The contractor shall remove all falsework, excavated material or useless material, replace or renew any fences damaged and leave the work in a condition satisfactory to the engineer. All excavated material or falsework placed in the stream channel during construction shall be removed by the contractor before final acceptance of the work.

All old material taken from the old bridge or excess material belonging to the purchaser shall be neatly piled or stored as directed by the engineer.

The engineer shall make final inspection upon being notified by the contractor that the work is completed. If the work is not acceptable to the engineer he shall advise the contractor as to the particular defects that must be remedied.

# CHAPTER XXV.

## Estimates and Costs of Highway Bridges and Culverts.

**Introduction.**—The cost of an engineering structure may be separated into several items as follows: (1) cost of plain materials; (2) cost of shop fabrication; (3) cost of engineering and inspection; (4) cost of transportation; (5) cost of erection. Before the estimate of cost can be prepared it will be necessary to prepare an estimate of the quantities. The estimates of steel bridges and of concrete bridges and culverts will be discussed separately.

**ESTIMATES OF WEIGHT OF STEEL HIGHWAY BRIDGES.**—There are three methods of estimating the weight of a steel bridge. (1) Estimate from finished shop drawings; (2) estimate from detail drawings; (3) estimate from stress sheet.

*ESTIMATE OF WEIGHT*

*132'-0" SPAN STEEL HIGHWAY BRIDGE*

*CODY, WYOMING*

*January 21, 1920* — *Sheet 1 of 11*

| Ref. No. | Number of Pieces | Shape | Section | Length Ft. | Length In. | Weight per Ft. Lb. | Weight Main Member Lb. | Weight Details Lb. | Details per cent Main Member | Total Weight Lb. |
|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 4 End-Post $L_0U_1$ each thus: | | | | | | | | | |
| | 2 | [s | 8"x11.25# | 27 | 6 | 11.25 | 619 | | | |
| | 1 | Cov. Pl. | 14"x$\frac{5}{16}$" | 27 | 8 | 14.88 | 412 | | | |
| | 2 | Bat. Pl. | 12"x$\frac{5}{16}$" | 1 | 2 | 12.75 | | 30 | | |
| | 4 | Pin Pl. | 6"x$\frac{5}{16}$" | 1 | 1$\frac{5}{8}$ | 6.38 | | 29 | | |
| | 4 | Pin Pl. | 6"x$\frac{5}{16}$" | 0 | 10$\frac{3}{8}$ | 6.38 | | 22 | | |
| | 31 | Lacing Bars | 1$\frac{1}{2}$"x$\frac{1}{4}$" | 1 | 4$\frac{11}{16}$ | 1.28 | | 55 | | |
| | 504 | Rivet Heads | $\frac{5}{8}$"$\phi$ | per 100 | | 8.60 | | 43 | | |
| | | | | | | | 1031 | 179 | 17.3 | |
| | | | | | | | 4x | 1210 = | | 4840 |

Fig. 1.

(1) **Estimate from Shop Drawings.**—In making an estimate of a steel highway bridge from the shop drawings the form shown in Fig. 1, will be found very convenient. The weights of the members in the order end-posts, top chords, lower chords, intermediate posts, main ties, hip verticals, counter ties, floorbeams, joists, hub guard, wall plates, top lateral struts, sway struts, top lateral rods, sway rods, bottom lateral rods, portals, chord pins and nuts, pedestals, bolts and spikes are calculated. Details of the calculation of the weight of the end-posts of the 132-ft. span pin-connected highway bridge shown in Fig. 8 to Fig. 10 in Chapter XIV are given in Fig. 1. The summary of the total weight of the metal in the bridge calculated from the shop drawings

is given in Table II. The "main members" are those that are given on the stress sheet and are either members in which stresses occur or which are specified by the designing engineer; while the "details" are plates, angles, rivets, etc., which are necessary to develop the strength of the main members. The values given in column 10 are the weights of "details" in per cent of weights of "main members." The weights per foot given in column 7 were obtained from Ketchum's "Structural Engineer's Handbook," or from Cambria or Carnegie. The weight for rivet heads should be the mean of the weight of rivet heads as made on the rivet and as driven in work. The actual shipping weight is desired, and the weights of rivet heads, only, are calculated, it being assumed that the remainder of the rivets fill the holes punched in the members. The total weights of the different parts of the bridge and the percentage of details are shown in Table II. The total weight of details in per cent of main members, exclusive of fence, joists, wall plates, bolts for lumber and spikes, is 35.4 per cent. The total weight of rivet heads in per cent of total weight of bridge, exclusive of fence, joists, etc., is 4.5 per cent.

TABLE I.

SUMMARY OF WEIGHT OF METAL.

111 ft. 6 in. × 18 ft. 0 in., Riveted Highway Bridge.

| Ref. No. | Member. | Weights, Lb. | | | Details Per Cent of Main Members. |
|---|---|---|---|---|---|
| | | Main. | Details. | Total. | |
| 1 | End-posts | 5,592 | 3,892 | 9,484 | 67.0 |
| 2 | Top chords | 5,900 | 3,942 | 9,842 | 67.0 |
| 3 | Lower Chords | 5,232 | 442 | 5,674 | 8.5 |
| 4 | Intermediate Posts | 2,436 | 2,835 | 5,277 | 116.0 |
| 5 | Main Ties | 3,184 | 474 | 3,658 | 15.0 |
| 6 | Hip Verticals | 856 | 163 | 1,019 | 19.0 |
| 7 | Counters | 1,156 | 109 | 1,265 | 9.0 |
| 8 | Floorbeams | 8,350 | 2,230 | 10,580 | 27.0 |
| 12 | Struts | 1,486 | 544 | 2,030 | 36.0 |
| 13 | Top Laterals | 531 | 35 | 566 | 7.0 |
| 14 | Bottom Laterals | 843 | 182 | 1,025 | 21.0 |
| 15 | Portals | 1,732 | 620 | 2,352 | 36.0 |
| 16 | Pins and Nuts | | 86 | 86 | |
| 17 | Pedestals | | 1,949 | 1,949 | |
| | | 37,298 | 17,503 | 54,801 | 46.9 |
| Total Weight of Metal in Bridge, exclusive of 9, 10, 11, 18 and 19 = 54,801 lb. | | | | | |
| 9 | Joists | 23,852 | 2,200 | 26,052 | 9.0 |
| 10 | Hub Guard | 2,392 | 267 | 2,659 | 11.0 |
| 11 | End Struts | 469 | 167 | 636 | 36.0 |
| 18 | Bolts for Lumber | | 365 | 365 | |
| 19 | Spikes for Lumber | | 389 | 389 | |
| | | 26,713 | 3,388 | 30,101 | 13.0 |
| | Total Metal in Bridge | 64,011 | 20,891 | 84,902 | 33.0 |

The weight of pins in highway bridges varies from 2 to 3 per cent of the total weight of the metal, exclusive of joists, fence, etc. The weight of rivet heads in pin-connected bridges varies from 2 to 4½ per cent of the total weight of the metal, exclusive of joists, fence, etc. The weight of rivet heads in riveted highway bridges varies from 2.5 to 5 per cent of the total weight of the metal, exclusive of fence, joists, etc. The total weight of the rivet heads in the 111 ft. 6 in. riveted Pratt truss highway bridge, shown in Figs. 1 and 2, Chapter XIV, was 2.8 per cent of the total weight of the metal, exclusive of the joists, fence, etc.

The details in per cent of total weight of main members, exclusive of joists, fence, etc., will vary from 30 to 40 per cent for pin-connected and riveted highway bridges in which the top chords are made of two channels and one plate, of two angles placed back to back, or two angles and a plate; while for bridges with open chords composed of two channels laced, or two angles laced, the details will vary from 35 to 50 per cent of the weight of the main members. The details of low truss riveted highway bridges will weigh from 35 to 45 per cent of the total weight of main members, exclusive of joists, fence, etc.

The weight of pedestals in terms of the total weight of metal, exclusive of joists, fence, etc., will vary from 2 to 4 per cent. The summary of the weight of the 132-ft. span pin-connected bridge shown in Fig. 8 to Fig. 10, Chapter XIV, is given in Table II, while the weight of the details of the members of a 111 ft. 6 in. riveted Pratt highway bridge are given in Table I. The shop drawings of this bridge are shown in Figs. 1 and 2, Chapter XIV. This bridge has heavy details. As a rule, highway bridges do not have sufficient details to properly develop the strengths of the members. In this connection it should be remembered that the per cent of details should be larger for light country highway bridges than for heavy city or electric railway bridges designed under similar specifications.

The per cent of the details for different individual members of a bridge can be seen by the study of Table I and Table II. It will be seen that there is a great variation in the details depending upon the make-up of the member and other conditions. The estimater should work out numerous problems and in this manner develop his estimating sense.

TABLE II.

SUMMARY OF WEIGHT OF METAL.

132 ft. 0 in. × 15 ft. 2 in., Pin-connected Highway Bridge.

| Ref. No. | Member. | Weights, Lb. Main Members. | Details. | Total. | Details, Per Cent of Main Members. |
|---|---|---|---|---|---|
| 1 | End-posts | 4,124 | 892 | 5,016 | 21.5 |
| 2 | Top Chords | 7,420 | 1,766 | 9,186 | 23.8 |
| 3 | Lower Chords | 5,048 | 1,940 | 6,988 | 38.5 |
| 4 | Intermediate Posts | 3,492 | 1,902 | 5,394 | 54.5 |
| 5 | Main Ties | 3,852 | 1,116 | 4,968 | 29.0 |
| 6 | Hip Verticals | 744 | 540 | 1,284 | 72.5 |
| 7 | Counters | 564 | 92 | 656 | 16.3 |
| 8 | Floorbeams | 4,570 | 522 | 5,092 | 11.4 |
| 12 | Struts | 590 | 30 | 620 | 5.0 |
| 13 | Top Laterals | 496 | 67 | 563 | 13.5 |
| 14 | Bottom Laterals | 1,285 | 149 | 1,434 | 11.6 |
| 15 | Portals | 812 | 252 | 1,064 | 31.0 |
| 16 | Pins and Nuts | | 843 | 843 | |
| 17 | Pedestals | | 1,562 | 1,562 | |
| | | 32,997 | 11,673 | 44,670 | 35.4 |
| Total Weight of Metal in Bridge, exclusive of 9, 10, 11, 18 and 19 = 44,670 lb. | | | | | |
| 9 | Joists | 20,628 | | 20,628 | |
| 10 | Hub Guard | | | 0 | |
| 11 | End Struts | | | 0 | |
| 18 | Bolts and Washers | | 213 | 213 | |
| 19 | Spikes for Lumber | | 85 | 85 | |
| | | 20,628 | 298 | 20,926 | 1.5 |
| | Total Metal in Bridge | 53,625 | 11,971 | 65,596 | 22.3 |

(2) **Estimate from Detail Drawings.**—Detail drawings show the main members partially detailed. The drawings give the number and approximate sizes of plates, the sizes of lacing bars, rivets, etc., and the approximate rivet spacing. In making an estimate from detail drawings the main members are taken from the drawings, while part of the details are supplied by the estimater. In order that the estimate be accurate the esimater must be familiar with the shop standards of the company that will fabricate the structure.

(3) **Estimate from the Stress Sheet.**—In this method the weights of the main members are calculated directly from the stress sheet, while the weights of the details are supplied by the estimater. The weight of the details may be estimated (*a*) by adding a percentage to each member—end-post, top chord, etc., or (*b*) by adding a percentage to the total weight of main members, exclusive of fence, joists, etc. The second method is very satisfactory where a standard type of bridge is used, while the first method should always be used for new types of construction.

Approximate estimates may be obtained from calculated weights, as shown in Chapter IX. This method is quite accurate when the tables or diagrams have been calculated for the standards in use.

*Accuracy of Estimates.*—The rolls used in rolling sections are designed to give a section of the required weight when the rolls are new, so that sections are usually slightly heavier than the figured weights due to the wear or the spreading of the rolls. It is commonly specified that the actual weight of fabricated steel work may vary not more than 2½ per cent from the figured weight. This means that where fabricated structural steel is bought at a pound price, the purchaser will have to pay for the actual weight, providing it does not exceed the calculated weight by more than 2½ per cent. Where fabricated structural steel work is more than 2½ per cent lighter than the calculated weight, the purchaser may refuse to accept the material. This latter case never occurs unless sections lighter than those shown on the drawings are substituted. The estimate made from shop drawings should be used as a basis for comparison. The results obtained from the detail drawings or from stress sheets should not vary from shipping weight by more than 1½ to 2 per cent, and should be a little heavy rather than light. Estimates from stress sheets should be made only by a skilled estimater.

*Shop Waste.*—The shipping weight of fabricated structural steel will be less than the weight of the rolled steel, due to the loss in rivet slugs, clippings, beveled cuts, milling, etc. This loss will vary from 3 to 5 per cent for highway bridges.

**Estimate of Lumber.**—Lumber is estimated in board feet, a board foot being a piece 12 in. square and 1 in. thick. Commercial sizes of lumber are less than the stated dimensions, so that where full sized timbers are desired it is necessary to specify this explicitly. Specifications for bridge timbers are given in Chapter XVI.

*Weight of Floor.*—The weight of oak is commonly taken as 4½ and pine 3½ lb. per foot B. M. The actual weights of other materials should be calculated, see Appendix I.

**ESTIMATE OF COST.**—The cost of a steel highway bridge may be divided into (1) cost of material, (2) cost of fabrication, (3) cost of transportation, (4) cost of erection, (5) cost of substructure, and (6) profit. The subject of costs is a very difficult matter to handle, and the author would caution the reader to use the data given on the following pages with great care, for the reason that costs are always relative and what may be a fair cost in one case may be sadly in error in another case, which appears to be an exact parallel. The price of labor will be given in each case *or the cost will be charged on the basis of* 40 *cents per hour*, which includes labor, cost of management, tools, etc. The costs given below are the average costs for a shop with a capacity of about 1,000 tons per month that has made a specialty of highway bridge work. *The costs given are based on a charge of* 40 *cents per hour for the number of hours actually consumed in getting out the contract.* This charge is assumed to cover the cost of management, cost of operation and maintenance, as well as the cost of labor. The cost of management in a small shop is very low, but in a large concern it may amount to as much as 35 to 40 per cent of all the other charges

combined. For this reason small shops can often fabricate light highway bridge steel for a less cost than the large shops. For additional data on the costs of structural steel, see the author's "The Design of Steel Mill Buildings," "The Design of Walls, Bins and Grain Elevators," "The Design of Mine Structures" and "The Structural Engineers' Handbook."

The costs given for labor are based on scales of labor in 1913 and represent prewar conditions. Before applying these labor costs to present structures, the engineer should make the proper adjustment for the increase in wages and also for the present decrease in efficiency of workmen.

1. **Cost of Material.**—The price of structural steel is quoted in cents per pound delivered f. o. b. cars at the point at which the quotation is made. Current prices may be obtained from the Engineering News-Record, Iron Age or other technical papers. The present prices (November, 1919) f. o. b. Pittsburgh, Pa., are about as follows:

TABLE III.

PRICES OF STRUCTURAL STEEL (1919) F. O. B. PITTSBURGH, PA., IN CENTS PER POUND.

| Material | Price in Cts. per Lb. |
|---|---|
| I-beams, 18 in. and over | 2.55 |
| I-beams and channels, 3 in. to 15 in. | 2.45 |
| H-beams, over 8 in. | 2.60 |
| Angles, 3 in. to 6 in. inclusive | 2.45 |
| Angles, over 6 in. | 2.50 |
| Zees, 3 in. and over | 2.45 |
| Angles, channels, and zees, under 3 in. | 2.40 |
| Deck beams and bulb angles | 2.75 |
| Checkered and corrugated plates | 2.75 to 2.90 |
| Plates, structural, base | 2.66 |
| Plates, flange, base | 2.76 |
| Corrugated steel No. 22, painted | 4.50 |
| Corrugated steel No. 22, galvanized | 5.50 |
| Steel sheets Nos. 10 and 11, black | 3.55 |
| Steel sheets Nos. 10 and 11, galvanized | 4.70 |
| Steel sheets No. 22, black | 4.20 |
| Steel sheets No. 22, galvanized | 5.30 |
| Bar iron, base | 2.45 |
| Rivets | 4.20 |

**Round and Square Bars.**—In estimating round and square bars use the standard card for extras, Table IV. It is not usual to enforce more than one-half the standard card extras for round and square bars.

**Extras.**—*Shapes, Plates and Bars:*

(Cutting to length)

| | |
|---|---|
| Under 3 ft. to 2 ft., inclusive | 0.25 ct. per lb. |
| Under 2 ft. to 1 ft., inclusive | 0.50 ct. per lb. |
| Under 1 ft. | 1.55 ct. per lb. |

*Extras—Plates* (*Card of January* 7, 1902):

Base ¼ in. thick, 100 in. wide and under, rectangular (see sketches).

| | Per 100 Lb. |
|---|---|
| Widths—100 in. to 110 in. | $ .05 |
| 110 in. to 115 in. | .10 |
| 115 in. to 120 in. | .15 |
| 120 in. to 125 in. | .25 |
| 125 in. to 130 in. | .50 |
| Over 130 in. | 1.00 |
| Gages under $\frac{1}{4}$ in. to and including $\frac{3}{16}$ in. | 10 |
| Gages under $\frac{3}{16}$ in. to and including No. 8 | .15 |
| Gages under No. 8 to and including No. 9 | .25 |
| Gages under No. 9 to and including No. 10 | .30 |
| Gages under No. 10 to and including No. 12 | .40 |
| Complete circles | .20 |
| Boiler and flange steel | .10 |
| Marine and fire box | .20 |
| Ordinary sketches | .10 |

(Except straight taper plates, varying not more than 4 in. in width at ends, narrowest end not less than 30 in., which can be supplied at base prices.)

## TABLE IV.

### STANDARD CLASSIFICATION OF EXTRAS ON IRON AND STEEL BARS.*

*Rounds and Squares.*

Squares up to $4\frac{1}{2}$ inches only. Intermediate sizes take the next higher extra.

| | Per 100 Lb. |
|---|---|
| $\frac{3}{4}$ to 3 in. | Rates. |
| $\frac{5}{8}$ to $\frac{11}{16}$ " | $0.10 extra. |
| $\frac{1}{2}$ to $\frac{9}{16}$ " | .20 " |
| $\frac{7}{16}$ " | .40 " |
| $\frac{3}{8}$ " | .50 " |
| $\frac{5}{16}$ " | .70 " |
| $\frac{1}{4}$ and $\frac{9}{32}$ " | 1.00 " |
| $\frac{7}{32}$ " | 2.00 " |
| $\frac{3}{16}$ " | 2.50 " |
| $3\frac{1}{16}$ to $3\frac{1}{2}$ " | .15 " |

*Flat Bars and Heavy Bands.*

| | |
|---|---|
| $3\frac{9}{16}$ to 4 in. | .25 extra. |
| $4\frac{1}{16}$ to $4\frac{1}{2}$ " | .30 " |
| $4\frac{9}{16}$ to 5 " | .40 " |
| $5\frac{1}{8}$ to $5\frac{1}{2}$ " | .50 " |
| $5\frac{5}{8}$ to 6 " | .75 " |
| $6\frac{1}{8}$ to $6\frac{1}{2}$ " | 1.00 " |
| $6\frac{5}{8}$ to $7\frac{1}{2}$ " | 1.25 " |
| 1 to 6 in. × $\frac{3}{8}$ to 1 in. | Rates. |
| 1 to 6 " × $\frac{1}{4}$ and $\frac{5}{16}$ " | $0.20 extra. |
| $\frac{11}{16}$ to $\frac{15}{16}$ " × $\frac{3}{8}$ to $\frac{3}{4}$ " | .40 " |
| $\frac{11}{16}$ to $\frac{15}{16}$ " × $\frac{1}{4}$ and $\frac{5}{16}$ " | .50 " |

* This classification has been quite generally adopted, although several firms issue a special card of extras. Carnegie Steel Company's (1919) extras are about one-half of the values in Table IV.

TABLE IV—*Continued.*

STANDARD CLASSIFICATION OF EXTRAS ON IRON AND STEEL BARS.

*Flat Bars and Heavy Bands.*

| | |
|---|---|
| 3/16 and ⅝ in. × ⅜ to ½ in. .......... | .50 extra. |
| 3/16 and ⅝ " × ¼ and 3/16 " .......... | .70 " |
| ½ " × ⅜ and 7/16 " .......... | .90 " |
| ½ " × ¼ and 5/16 " .......... | 1.10 " |
| 7/16 " × ⅜ " .......... | 1.00 " |
| 7/16 " × ¼ and 5/16 " .......... | 1.20 " |
| ⅜ " × ¼ and 3/16 " .......... | 1.50 " |
| 1⅛ to 6 in. × 1 1/16 to 1 3/16 in. .......... | .10 " |
| 1⅛ to 6 " × 1¼ to 1½ " .......... | .20 " |
| 1¾ to 6 " × 1⅝ to 2¾ " .......... | .30 " |
| 3⅛ to 6 " × 3 to 4 " .......... | .40 " |

**Mill Orders.**—In mill orders the following items should be borne in mind. Where beams butt at each end against some other member, order the beams ½ in. shorter than the figured lengths, this will allow a clearance of ⅛ in. if all beams come ⅜ in. too long. Where beams are to be built into the wall, order them in full lengths, making no allowance for clearance. Order small plates in multiple lengths. Irregular plates on which there will be considerable waste should be ordered cut to templet. Mills will not make reentrant cuts in plates. Allow ⅛ in. for each milling for members that have to be faced. Order web plates for girders ⅛ to ½ in. narrower than the distance back to back of angles. Order as nearly as possible every thing cut to required length, except where there is liable to be changes made, in which case order long lengths.

It is often possible to reduce the cost of mill details by having the mills do only part of the work, the rest being done in the field, or by sending out from the shop to be riveted on in the field connection angles and other small details that would cause the work to take a very much higher price. Standard connections should be used wherever possible, and special work should be avoided.—For additional notes on ordering material, see Chapter XV.

(*b*) **COST OF MILL DETAILS.**—If material is ordered directly from the rolling mill the price for the necessary cutting to exact length, punching, etc., is based on a standard "card of mill extras."

**CARD OF MILL EXTRAS.**—If the estimate is to be based on card rates it will be necessary to have the subdivisions a, b, c, d, e, f, r, etc., as follows:

*a* = 0.15*cts. per lb.* This covers plain punching one size of hole in web only. Plain punching one size of hole in one or both flanges.

*b* = 0.25*cts. per lb.* This covers plain punching one size of hole either in web and one flange or web and both flanges. (The holes in the web and flanges must be of same size.)

*c* = 0.30*cts. per lb.* This covers punching of two sizes of holes in web only. Punching of two sizes of holes either in one or both flanges. One size of hole in one flange and another size of hole in the other flange.

*d* = 0.35*cts. per lb.* This covers coping, ordinary beveling, riveting or bolting of connection angles and assembling into girders, when the beams forming such girders are held together by separators only.

*e* = 0.40*cts. per lb.* This covers punching of one size of hole in the web and another size of hole in the flanges.

*f* = 0.15*cts. per lb.* This covers cutting to length with less variation than ± ⅜ in.

*r* = 0.50*cts. per lb.* This covers beams with cover plates, shelf angles, and ordinary riveted beam work. If this work consists of bending or any unusual work, the beams should not be included in beam classification.

**Fittings.**—All fittings, whether loose or attached, such as angle connections, bolts, separators, tie rods, etc., whenever they are estimated in connection with beams or channels to be charged at 1.55cts. per lb. over and above the base price. The extra charge for painting is to be added to the price for fittings also. The base price at which fittings are figured is not the base price of the beams to which they are attached but is in all cases the base price of beams 15 in. and under.

The above rates will not include painting, or oiling, which should be charged at the rate of 0.10cts. per lb. for one coat, over and above the base price plus the extra specified above.

For plain punched beams where more than two sizes of holes are used, 0.15cts. per lb. should be added for each additional size of hole, for example, plain punched beams, where three sizes of holes occur would be indicated as: $c + 0.15$cts., four sizes of holes; $e + 0.30$cts. For example: a beam with $\frac{5}{8}$ in. and $\frac{3}{4}$ in. holes in the flanges and $\frac{7}{8}$ in. and $\frac{1}{2}$ in. holes in the web should be included in class *e*.

Cutting to length can be combined with any of the other rates, class *d* excepted, and would have to be indicated; for example: Plain punching one size of hole in either web and one flange, or web and both flanges, and cutting to length would be marked *bf*, which would establish a total charge of 0.40cts. per lb.

*Note to class d.*—No extra charge can be added to this class for punching various sizes of holes, or cutting to exact lengths; in other words; if a beam is coped or has connection angles riveted or bolted to it, it makes no difference how many sizes of holes are punched in this beam, the extra will always be the same, namely 0.35cts. When beams have angles or plates riveted to them, and same are not half length of the beam, figure the beams as class *d*, and the plates and angles as beam connections.

*Note to class r.*—This rate of 0.50cts. per lb. applies to all the material making up the riveted beam. In case of assembled girders in which one of the beams should be classed as a riveted beam, in making up the estimate, figure only the beam affected as included in class "*r*." When beams have angles or plates riveted to them and same are half length or more than half length of the beam, figure the beams as class "*r*," including the plates or angles and rivets. When 18 in., 20 in., or 24 in. beams are in "*r*" class keep the I's separate from the material (plates, cast iron, separators, angles and rivets) which should go under heading, "15 in. I's and Under."

Beams should be divided as 15 in. I's and under, and 18 in., 20 in. and 24 in. I's. If there are only one or two sizes of beams in any particular class, give exact sizes, instead of "15 in. I's and Under."

In estimating channel roof purlins classify 7 in. channels and smaller as one punched; 8 in. channels and larger as two punched, unless they are shown or noted otherwise, and keep separate from other beams.

No extra charge can be added to curved beams for riveting, cutting to length, etc.

Subdividing work into a large number of classes should be avoided; it is better to have too few classes, rather than too many.

The only subdivision necessary for cast iron columns are: 1 in. and over, and under 1 in. Columns with ornamental work cast on must be kept separate.

In estimating the cost of plain material in a finished structure the shipping weight from the structural shop is wanted. The cost of material f. o. b. the shop must therefore include the cost of waste, paint material, and the freight from the mill to the shop. The waste is variable but as an average may be taken at 4 per cent. Paint material may be taken as two dollars per ton. The cost of plain material at the shop would be

| | |
|---|---|
| Average cost per lb. f. o. b. mill, say | 2.75 cts |
| Add 4 per cent for waste | .11 " |
| Add $2.00 per ton for paint material | .10 " |
| Add freight from mill to shop (Pittsburgh to St. Louis) | .24 " |
| Total cost per pound f. o. b. shop | 3.20 cts |

To obtain the average cost of steel per pound multiply the pound price of each kind of material by the percentage that this kind of material is of the whole weight, the sum of the products will be the average pound price.

(*c*) **COST OF SHOP LABOR.***—The cost of shop labor may be calculated for the different parts of the structure, or may be calculated for the structure as a whole. *The following costs are based on an average charge of* 40 *cents per hour and include detailing and shop labor.* The cost of fabricating beams, channels and angles which are simply punched or have connection angles loose or attached should be estimated on the basis of mill details, which see.

For cost of making shop details, see the author's "Structural Engineers' Handbook."

* With the present (1919) condition of the labor market the author thought it wise to reprint the costs of shop labor given in the author's "Structural Engineer's Handbook." These costs were based on conditions in 1913, and represented prewar conditions.

**SHOP COSTS OF INDIVIDUAL PARTS OF BRIDGES.**—The cost of fabricating joists and other similar members should be estimated on the basis of mill details, which see. (Based on prewar conditions in 1913 and a cost 40 cts. per hour which includes detailing and shop labor).

**Eye-Bars.**—The shop cost of eye-bars varies with the size and length of the bars and the number made alike. The following costs are a fair average: Average shop costs of bars 3 in. and less in width and $\frac{3}{4}$ in. and less in thickness is from 1.20 to 1.80 cts. per lb., depending upon the length and size. A good order of bars running $2\frac{1}{2}$ in. × $\frac{3}{4}$ in. to 3 in. × $\frac{3}{4}$ in., and from 16 to 20 ft. long, with few variations in size, will cost about 1.20 cts. per lb. Large bars in long lengths ordered in large quantities can be fabricated at from 0.55 to 0.75 cts. per lb. To get the total cost of eye-bars the cost of bar steel must be added to the shop cost. Half card extras given in Table IV should ordinarily be added to the base price of plain steel bars.

**Chords, Posts and Towers.**—In lots of at least four, the shop cost is about as follows: Members made of two channels and a top cover plate with lacing on the bottom side, or two channels laced on both sides cost about 1.00 to 0.85 cts. per lb. for pin-connected members weighing from 600 to 1,500 lb.; and about 0.80 to 0.70 cts. per lb. for members with riveted end connections. Members made of four angles laced cost from 0.80 to 1.10 cts. per lb. for members with riveted ends. Members made of two angles battened will cost about 0.50 cts. per lb. Angles used without end connections should have their cost estimated on the basis of mill details, which see.

**Pins.**—The cost of chord pins will vary with the size, number and other requirements. The shop cost of chord pins and nuts may be estimated at from 2.00 to 3.00 cts. per lb. Rollers will cost practically the same as pins. Rolled rounds (pin rounds) are used for making pins and rollers.

**Latticed Fence.**—The shop cost of light simple latticed fence made of two 2 in. × 2 in. angles, with double lacing and about 18 in. deep, will be about 2.00 cts. per lb.; while the shop cost of latticed fence, with ornamental rosettes or ornamental plates, may be as much as 4.00 to 5.00 cts. per lb.

**Floorbeams and Stringers.**—Plate girders used for floorbeams and stringers will cost from 0.60 to 1.25 cts. per lb. depending upon the weight, details and number made at one time. Floorbeams made of rolled I-beams will cost from 0.50 to 0.75 cts. per lb.

**SHOP COSTS OF BRIDGES AS A WHOLE.**—The cost will be taken up under the head of pin-connected bridges, riveted bridges, plate girder bridges, combination bridge metal, and Howe truss metal. These shop costs represent prewar conditions in 1913, and were based on an average charge of 40 cents per hour and include detailing and shop labor.

**Shop Costs of Pin-connected Bridges.**—The shop costs of pin-connected highway or railway bridges, exclusive of fence and joists, are about as follows:

| | | | |
|---|---|---|---|
| Bridges weighing | 5,000 lb. and less | 1.30 | cts. per lb. |
| " " | 5,000 to 10,000 lb. | 1.20 | " " " |
| " " | 10,000 to 20,000 lb. | 1.00 | " " " |
| " " | 20,000 to 40,000 lb. | 0.90 | " " " |
| " " | 40,000 to 60,000 lb. | 0.80 | " " " |
| " " | 60,000 to 100,000 lb | 0.75 | " " " |
| " " | 100,000 to 150,000 lb. | 0.70 | " " " |
| " " | 150,000 and up | 0.65 | " " " |

These costs include detailing and one coat of shop paint. For reaming add 0.15 cts. per lb.

**Shop Costs of Riveted Truss Bridges.**—The shop costs of riveted truss highway or railway bridges, exclusive of fence and joists, are about as follows:

| | | |
|---|---|---|
| Bridges weighing | 5,000 lb. and less | 1.15 cts. per lb. |
| " " | 5,000 to 10,000 lb. | 1.00 " " " |
| " " | 10,000 to 20,000 lb. | 0.90 " " " |
| " " | 20,000 to 40,000 lb. | 0.85 " " " |
| " " | 40,000 to 60,000 lb. | 0.75 " " " |
| " " | 60,000 to 100,000 lb. | 0.70 " " " |
| " " | 100,000 to 150,000 lb. | 0.65 " " " |
| " " | 150,000 lb. and up | 0.60 " " " |

These costs include detailing and one coat of shop paint. For reaming add 0.15 cts. per lb.

**Shop Costs of Plate Girder Bridges.**—The shop costs of plate girder highway or railway bridges, exclusive of fence and joists, are about as follows:

| | | |
|---|---|---|
| Spans weighing | 10,000 lb. and less | 0.90 cts. per lb. |
| " " | 10,000 to 20,000 lb. | 0.85 " " " |
| " " | 20,000 to 40,000 lb. | 0.75 " " " |
| " " | 40,000 to 60,000 lb. | 0.70 " " " |
| " " | 60,000 to 100,000 lb. | 0.60 " " " |
| " " | 100,000 and up | 0.50 " " " |

These costs include detailing and one coat of shop paint. For reaming add 0.15 cts. per lb.

**Shop Costs of Tubular Piers and Culverts.**—The shop costs of steel tubular pier shells and steel culvert pipe are about as follows:

| | | | |
|---|---|---|---|
| Tubes | 18 in. to 24 in. diameter, | ¼ in. metal | 1.00 cts. per lb. |
| " | 24 in. to 30 in. diameter, | ¼ in. to ⅜ in. metal | 0.75 to 0.65 " " " |
| " | 30 in. to 48 in. diameter, | ¼ in. to ⅜ in. metal | 0.70 to 0.60 " " " |
| " | 48 in. to 72 in. diameter, | ¼ in. to ½ in. metal | 0.65 to 0.50 " " " |
| " | 72 in. and up | ⅜ in. to ⅝ in. metal | 0.50 to 0.45 " " " |

The above shop costs include detailing and one coat of shop paint. The necessary bracing and rods for tubular piers are included.

**Shop Cost of Combination Bridge Metal.**—Where the bars and rods are standard and the castings are made from standard patterns, the metal for combination bridges can be fabricated at about the same cost per pound as for pin-connected spans weighing the same as the weight of the metal in the combination bridges.

**Shop Cost of Howe Truss Bridge Metal.**—The shop cost of highway bridge castings made from standard patterns, is from 1.50 to 2.00 cts. per lb. The shop costs of the plates, rods and other miscellaneous iron work will be from 2.00 to 2.50 cts. per lb.

**COST OF ERECTION OF STEEL BRIDGES.***—The cost of erection ordinarily includes: (1) the cost of hauling the bridge to the bridge site; (2) the building of the falsework and the placing of the steel in position; (3) the riveting up of the bridge, and (4) painting the steel and the woodwork.

**Hauling.**—Transportation over country roads will ordinarily cost about 25 cts. per ton-mile, in addition to the cost of loading and unloading. In estimating the cost of hauling on any particular job the length of haul, kind of roads, price of teams and labor, and the character of the teams should be considered. The cost of loading on the wagons and unloading will depend upon the local conditions, but will ordinarily be from 25 to 50 cts. per ton. For railroad bridges the steel work may ordinarily be brought directly to the site by rail.

* Based on prewar conditions in 1913.

**Falsework.**—If piles are to be used the cost snould be carefully estimated. The cost of the piles in place will vary with the cost of piles and local conditions. Under ordinary conditions piles in falsework will cost from 25 to 50 cts. per lineal foot in place. The cost of the timber will depend upon local conditions and upon what use is made of it after erection. The flooring plank in highway bridges, and ties and guard timbers in railway bridges can often be used in the falsework without serious injury. The cost of erecting the timber in the falsework will ordinarily be from $6.00 to $8.00 per thousand ft. B. M.

**Erection of Tubular Piers.**—The cost of setting tubular piers for highway bridges will depend upon the conditions. Tubes 36 in. in diameter and 20 ft. long have been set in favorable locations for $25.00 per pair, not including the driving of the piles or the placing of the concrete. It is, however, not safe to estimate the cost of setting tubes from 36 to 48 in. in diameter under even favorable conditions at less than $2.00 per lineal foot of tube. When the cost of setting tubes is estimated by weight, it should be figured at from $15.00 to $20.00 per ton, for ordinary conditions. It will commonly cost from 25 to 50 cts. per lineal ft. to drive piles in tubes, in addition to the cost of the piles, which will vary from 10 to 20 cts. per lineal foot. The concrete will commonly cost from $6.00 to $8.00 per cu. yd. in place in the tube.

**Placing and Bolting.**—The cost of placing and bolting up riveted highway spans, and erecting pin-connected highway spans, no rivets being driven, is about as follows:

| | | |
|---|---|---|
| Highway spans from 30 to 60 ft. | $12.00 to $15.00 | per ton. |
| " " " 60 to 100 ft. | 10.00 to 12.00 | " " |
| " " " 100 to 150 ft. | 9.00 to 10.00 | " " |
| " " " 150 ft. and up | 8.00 | " " |

The cost of placing and bolting up railroad spans will depend so much upon the local conditions and equipment that it is difficult to give general costs.

The cost of driving field rivets in pin-connected spans will vary from 7 to 12 cts. per rivet, while the cost of driving field rivets in riveted trusses will vary from 6 to 10 cts. per rivet. The number of rivets in riveted low truss highway bridges depends upon the number of panels and the style of details, and will be about 155 to 200 for a three-panel bridge, and 400 to 500 for a six-panel bridge. The number or rivets in through riveted highway bridges will be about 250 to 300 for a four-panel bridge, and 1,300 to 1,500 for a nine-panel bridge. Pin-connected bridges ordinarily have about ½ to ⅓ as many field rivets as a riveted bridge of similar dimensions.

**Transportation.**—Fabricated structural steel commonly takes a "fifth-class rate" when shipped in car load lots, and a "fourth-class rate" when shipped "local" (in less than car load lots). The minimum car load depends upon the railroad and varies from 20,000 to 30,000 lb. Tariff sheets giving railroad rates may be obtained from any railroad company. The shipping clerk should be provided with the clearances of all tunnels and bridges on different lines so that the car may be properly loaded.

**Freight Rates.**—The freight rates (1919) on finished steel products in car load shipments from the Pittsburgh District, including plates, structural shapes, merchant steel and iron bars, pipe fittings, plain and galvanized wire, nails, rivets, spikes and bolts (in kegs), black sheets (except planished), chain, etc., are as follows, in cts. per 100 lb. in carload shipments; Albany, 30; Buffalo, 17; Chicago, 27; Cincinnati, 23; Cleveland, 17; Denver, 99; Kansas City, 59; New Orleans, 38½; New York, 27; Pacific Coast (all rail), 125; Philadelphia, 24½; St. Louis, 24; St. Paul, 49½; Detroit, 33; Baltimore, 33. (Add 3 per cent transportation tax).

**COST OF PAINTING.**—The amount of materials required to make a gallon of paint and the surface of steel work covered by one gallon are given in Table V. Structural steel should be painted with one coat of linseed oil, linseed oil with lamp-black filler, or red lead paint at the shop; and two coats of first-class paint after erection. The two field coats should be of

different colors; care being used to see that first coat is thoroughly dry before applying the second coat. Steel bridges and exposed steel frame buildings ordinarily require repainting every three or four years.

The steel work in the extension to the 16th St. Viaduct, Denver, Colo., was painted with red lead paint mixed in the following proportions,—100 lb. red lead, 2 lb. lamp-black and 4.125 gallons of linseed oil. This mixture made 6 gallons of mixed paint of a chocolate color, and gave 1.455 gallons of paint for each gallon of oil.

TABLE V.

AVERAGE SURFACE COVERED PER GALLON OF PAINT.
PENCOYD HAND BOOK.

| Paint. | Volume of Oil. | Pounds of Pigment. | Volume and Weight of Paint. | | Square Feet. | |
|---|---|---|---|---|---|---|
| | | | Gal. | Lb. | 1 Coat. | 2 Coats. |
| Iron oxide (powdered) | 1 gal. | 8.00 | 1.2 = | 16.00 | 600 | 350 |
| Iron oxide (ground in oil) | 1 gal. | 24.75 | 2.6 = | 32.75 | 630 | 375 |
| Red lead (powdered) | 1 gal. | 22.40 | 1.4 = | 30.40 | 630 | 375 |
| White lead (ground in oil) | 1 gal. | 25.00 | 1.7 = | 33.00 | 500 | 300 |
| Graphite (ground in oil) | 1 gal. | 12.50 | 2.0 = | 20.50 | 630 | 350 |
| Black asphalt | 1 gal. (turp.) | 17.50 | 4.0 = | 30.00 | 515 | 310 |
| Linseed oil (no pigment) | 1 gal. | ...... | ...... | ...... | 875 | ...... |

Light structural work will average about 250 sq. ft., and heavy structural work about 150 sq. ft. of surface per net ton of metal, while No. 20 corrugated steel has 2,400 sq. ft. of surface.

It is the common practice to estimate $\frac{1}{2}$ gallon of paint for the first coat and $\frac{3}{8}$ gallon for the second coat per ton of structural steel, for average conditions.

A good painter should paint 1,200 to 1,500 sq. ft. of plate surface or corrugated steel or 300 to 500 sq. ft. of structural steel work in a day of 8 hours; the amount covered depending upon the amount of staging and the paint. A thick red lead paint mixed with 30 lb. of lead to the gallon of oil will take fully twice as long to apply as a graphite paint or linseed oil. The cost of applying paint is roughly equal to the cost of a good quality of paint, the cost per ton depending on the spreading qualities of the paint. This rule makes the cost of applying a red lead paint with 30 lb. of pigment per gallon of oil from two to three times the cost of applying a good graphite paint, per ton of structural steel. For additional data on paints, see Chapter XV.

**Cost of Painting Steel Highway Bridges.***—The cost of painting two steel highway bridges in Iowa in 1918 by day labor was as follows. Bridges were 110 ft. span with timber joists. Labor, one man at 40ct. and one man at 35ct. per hour, $157.50. Twenty-five gallons blue paint, $53.75. Ten gallons white paint, $22.50. One and one-fourth gallons linseed oil, $2.25. Brushes, $8.18. Total cost $244.18. Cost per lineal foot of bridge, $1.11 for one coat.

Eight old highway bridges were painted by contract. The bridges were painted two coats. The county furnished the paint, paint brushes, and the contractor furnished all the work including the cleaning of the metal with wire brushes. The cost of painting four bridges having spans of 190 ft., 170 ft., 160 ft. and 140 ft., all on steel tubular piers was 85 ct. per lineal foot of bridge for labor. Wood guard rails and steel tubular piers were included in cost per lineal foot of span. Bridges on abutments cost for labor to paint two coats from 55 to 70 ct. per lineal foot.

**Cost of Repainting Old Steel Highway Bridges.**—The Iowa Highway Commission published the following data in its Service Bulletin in 1918.

One gallon good quality paint will cover 1,100 sq. ft. of steel surface or about 5 tons fabricated metal; 1 gal. of sublimed white or blue lead paint will cover 700 sq. ft. of steel surface, or 3.5 tons of fabricated metal. The cost of sand blasting to remove old paint is about $1.50 per ton of metal which includes a rental charge on equipment. Sand blasting equipment costs from $500 to $700.

* Engineering and Contractor, Nov. 13, 1918.

**Cold Bituminous Mixture on Timber Highway Bridge Floor.***—The Shippingsport bridge near La Salle, Illinois was covered with a bituminous mixture in 1916. The bituminous coat was made by mixing one gallon of emulsified asphalt per cubic foot of aggregate consisting of ¾ in. stone chips crushed fine with the dust removed and 20 per cent concrete sand added. Asphalt and sand mixed in ordinary batch mixer. Average thickness of mastic 1½ inch. Total square yards of surface treated 1,146. The cost per square yard not including contractors profit was, labor preparing floor including superintendence 26 cts. Stone and sand 17 cts., emulsified asphalt 30 cts., mixing and placing 17 cts., total cost per sq. yd. 90 cts. Labor was paid 40 cts. per hour, teams 80 cts. per hour.

**Tar Mat on Old Timber Bridge Floor.**—An old timber floor of a bridge 1,700 ft. long at Stillwater, Minnesota was covered with tar and gravel in 1917 at a cost of 19 ct. per sq. yd. The plank was calked with lath and thoroughly washed. When floor was dry it was coated with a priming coat of Tarvia B. As soon as Tarvia B was absorbed a coat of Tarvia A, heated to 125 degrees F. was applied, and immediately covered with a layer of gravel which had passed a 1 in. screen. A second and third coat were applied at intervals of 10 days. After a year the coating was in good condition.

**ESTIMATED COST OF A RIVETED TRUSS HIGHWAY BRIDGE.**—A detailed estimate will be made of the 111′ 6″ riveted Pratt steel highway bridge over the Illinois and Mississippi Canal, the detail shop plans of which are given in Figs. 1 and 2, Chapter XIV. Date of estimate, 1908. Present (1919) prices of materials and labor will be much higher, but the method of calculation will be the same. A summary of the estimate of the weight of this bridge is given in Table I.

**Cost of Material.**—The cost of the steel will be estimated at the mill at Pittsburg, Pa.

*Bridge, Exclusive of Joists and Fence.*—The bridge is composed of beams, angles, bars, plates and pin rounds as given in the following table:

| Shape. | Weight, Pounds. | Per Cent. of Total Weight. | Cost Per Pound, Cents. | Percentage of Pound Cost, Cents. |
|---|---|---|---|---|
| Beams and Channels | 22,278 | 39 | 1.70 | 0.663 |
| Angles 3″ and under | 15,779 | 27 | 1.70 | 0.459 |
| Angles over 3″ | 1,021 | 2 | 1.60 | 0.032 |
| Bars | 11,585 | 20 | 1.60 | 0.320 |
| Plates | 6,222 | 11 | 1.60 | 0.176 |
| Pin Rounds | 506 | 1 | 2.00 | 0.020 |
| Total | 57,391 | 100 | | 1.670 |

| | | |
|---|---|---|
| The average cost of the steel at the mill | 1.670 | cts. per lb. |
| Waste in fabrication, 4 per cent | 0.067 | " " " |
| Paint material | 0.010 | " " " |
| Freight, Pittsburgh to Chicago | 0.165 | " " " |
| Average cost of the steel at the shop | 1.912 | " " " |

*Joists.*—The joists, end struts and hub guards (fence) will take the rate of 1.70 cts. per lb. at the mill.

| | | |
|---|---|---|
| The average cost of the steel at the mill | 1.700 | cts. per lb. |
| Waste in fabrication, 2 per cent | 0.034 | " " " |
| Paint material | 0.010 | " " " |
| Freight, Pittsburgh to Chicago | 0.165 | " " " |
| Average cost of the steel at the shop | 1.909 | " " " |

* Engineering and Contracting, Aug. 1, 1917.

**Shop Cost of the Steel in the Bridge, Exclusive of Fence, Joists, etc.—**

| | | |
|---|---|---|
| Average cost of steel at the shop | 1.912 | cts. per lb. |
| Shop cost, including drafting | 0.750 | " " " |
| Total shop cost | 2.662 | " " " |
| Freight, shop to railroad station near site | 0.100 | " " " |
| Total cost at railroad station | 2.762 | " " " |

**Shop Costs of Joists, Fence and End Struts.—**

| | | |
|---|---|---|
| Average cost of the steel at the shop | 1.909 | cts. per lb. |
| Shop cost, including drafting | 0.250 | " " " |
| Total shop cost | 2.159 | " " " |
| Freight, shop to railroad station near site | 0.100 | " " " |
| Total cost at railroad station | 2.259 | " " " |

**Erection.—**

| | |
|---|---|
| Hauling 43 tons 4 miles, @ 25 cts. per ton mile for hauling and 50 cts. per ton for loading and unloading | $ 64.50 |
| Falsework.—Twenty piles 35 ft. long @ 15 cts. per ft. | 105.00 |
| Driving 525 lin. ft. piling @ 25 cts. | 131.25 |
| Timber, 6,000 ft. B. M.—½ price—@ $12.00 | 72.00 |
| Placing timber, 6,000 ft. B. M. @ $8.00 | 48.00 |
| Labor erecting and bolting the steel, 30 days, labor @ $4.00 | 120.00 |
| Transportation of men and tools | 60.00 |
| Driving 1,500 field rivets @ 10 cts. | 150.00 |
| Labor, painting bridge 2 coats, 10 days @ $4.00 | 40.00 |
| Labor, erecting floor lumber, 12,000 ft. B. M. @ $4.00 | 48.00 |
| Total cost of erection | $838.75 |

**Summary of Cost of Superstructure.—**

| | |
|---|---|
| Steel, 57,393 lb. @ 2.762 cts. per lb. | $1,685.10 |
| Joists, fence, etc., 26,713 lb. @ 2.259 cts. | 603.45 |
| Lumber—yellow pine, 5,715 ft. B. M. @ $25.00 | 142.87 |
| Lumber—oak plank, 6,540 ft. B. M. @ $32.00 | 209.28 |
| Paint, 20 gallons @ $1.25 per gallon | 25.00 |
| Bolts for the floor, 400 lb. @ 3 cts. | 12.00 |
| Spikes for the floor, 400 lb. @ 3 cts. | 12.00 |
| Cost of erection | 838.75 |
| Total cost of the superstructure | $3,528.45 |

**Contract Price.—**

| | |
|---|---|
| Steel in place, 84,106 lb. @ 4 cts. per lb. | $3,364.24 |
| Yellow pine in place, 5,715 ft. B. M. @ $36.00 | 205.74 |
| Oak timber in place, 6,540 ft. B. M. @ $46.00 | 300.84 |
| Total contract price | $3,870.82 |
| Profit | $ 342.37 |

**COST OF MASONRY ABUTMENTS AND PIERS.**—The cost of masonry abutments and piers varies between wide limits, depending upon the cost of stone, cost of quarrying, cost of dressing, cost of laying, cost of mortar, cost of superintendence, cost of tools, cost of maintenance and depreciation of plant. Space will not permit a discussion of all the above items.

*Cost of Stone.*—The price of stone is usually quoted f. o. b. at the quarry, and varies with the stone and location.

*Cost of Quarrying.*—After the quarry has been opened in limestone, two-man stone for rubble wall can usually be quarried for from ¼ to ⅓ the cost of the daily wages of a quarry laborer per cu. yd. Stones ranging from ⅓ to 1 cu. yd., that have to be blasted, will cost per cu. yd. from ⅓ to 2 times the cost of the daily wages of one man. Dimension stones that have to be wedged out will cost twice as much as the large stones that can be blasted. This estimate is high for sandstone and low for granite.

*Cost of Dressing.*—Rubble is roughly scabbled when it is laid and there is no special charge for dressing. Dimension stones, if dressed to lay with quarry finish and fairly close joints, will cost from $2.00 to $3.00 per cu. yd. Bush-hammering costs about 50 cents per sq. ft.

*Cost of Laying.*—One mason and a helper can lay from 4 to 5 cu. yd. of small rubble in a day of 8 hours. If a derrick is necessary and some dressing required, one mason and a helper will lay only from 2 to 3 cu. yd. of heavy rubble or 1½ to 2 cu. yd. of dimension stone in a day of 8 hours.

*Cost of Mortar.*—The amount of mortar required varies with the specifications and the stone used. Rubble masonry is from 20 to 35 per cent mortar. Dimension stone masonry is from 10 to 15 per cent mortar. Knowing the cost of cement and sand, the cost of the mortar can be estimated.

*Miscellaneous Costs.*—The cost of superintendence, tools, maintenance and depreciation of plant, etc., can only be estimated on the particular work. These costs may vary from 5 to 20 per cent of the cost above.

**ESTIMATES OF CONCRETE HIGHWAY BRIDGES AND FOUNDATIONS.**—The making of estimates of reinforced concrete structures involves (a) the calculation of the quantities of cement, sand, coarse aggregate, reinforcing steel, lumber and other materials which constitute the structure; (b) the cost of the materials which are to be used in the structure, and (c) the cost of the labor necessary to fabricate and erect the structure.

**Estimate of Quantities.**—The quantities of materials should be calculated from the plans. Forms similar to those used for structural steel bridges should be used. The different items should be worked out in detail in order that the different classes of material may be separated for determining the costs.

**Estimate of Concrete.**—All concrete should be measured by the cubic foot or cubic yard, net measurement in place. All openings and voids should be deducted, but no allowance need be made for bevels, or for reinforcing steel. The amount of cement, sand and stone or gravel in a cubic yard of concrete will vary with the proportions, and character of the aggregates. If the amount of cement and aggregates in the concrete have not been determined by test, the quantities can be calculated with considerable accuracy by means of Fuller's rule.

*Fuller's Rule.*—The proportions of concrete materials should be stated in terms of the volume of the cement. The volume of one barrel or four bags of cement is taken as 4.0 cu. ft. and the sand and aggregate are measured loose. Concrete mixed one part cement, 2 parts sand, and 4 parts stone is commonly called 1 : 2 : 4 concrete. The proportions should be such that there should be more than enough cement paste to fill the voids in the sand, and more than enough mortar to fill the voids in the stone. With voids in sand and stone varying from 40 to 45 per cent, the quantities of the ingredients are closely given by Fuller's rule, where

$c$ = number of parts of cement;
$s$ = number of parts of sand;
$g$ = number of parts of gravel or stone.

Then $\dfrac{11}{c+s+g} = p$ = number of barrels of Portland cement required for one cu. yd. concrete.

$\dfrac{p \times s \times 4.0}{27}$ = number of cu. yd. sand required for one cu. yd. concrete.

$\dfrac{p \times g \times 4.0}{27}$ = number of cu. yd. gravel or stone required for one cu. yd. concrete.

The materials for one cu. yd. of 1 : 2 : 4 concrete will then be: Portland cement 1.57 barrels, sand 0.47 cu. yd., gravel or stone 0.94 cu. yd.

The proportions for plain walls commonly vary from 1 : 2½ : 5 to 1 : 3 : 6, while the proportions for reinforced walls vary from 1 : 2 : 4 to 1 : 2½ : 5.

**Estimate of Reinforcing Steel.**—Reinforcing bars should be taken off the plans in linear feet and reduced to weight in pounds. Allowance should be made for laps. Pipe sleeves, turnbuckles, bolts, nuts and special items should be listed separately. Wire cloth, expanded metal and similar reinforcement sold in sheets should be taken off in square feet, allowance being made for laps. The size of mesh and weight of steel should be stated.

**Estimate of Forms.**—Forms should be measured in square feet, taking all the area of concrete coming in contact with the forms. The thickness of lumber and surface finish required should be noted. The posts, sills, struts and bracing required to support the forms should be taken off separately. If lumber is to be used more than once this fact should be noted. Piles for falsework should be listed separately, noting the size and length required.

**Estimate of Surface Finish.**—The finish of concrete surfaces should be measured in square feet. Sidewalk finish should be measured in square feet. Sidewalk finish laid after the structure is complete should be listed separately.

**Cost of Materials.**—The cost of cement at the mill varies with market conditions. The present (1919) price of cement at the mill in carload lots is about $2.00 per barrel. When shipped in cloth bags an extra charge of 60 cents per bbl. is made for bags, which charge is refunded if the bags are returned to the mill in good condition. The cost of testing cement at the mill is about 5 cts. per bag. The cost of freight and cost of unloading and hauling to the bridge site must be calculated for each structure.

**Cost of Sand, Gravel and Broken Stone.**—The cost of sand depends upon local conditions and may be as low as $1.00 per cu. yd. if obtained locally and may be from $2.00 to $3.00 per cu. yd. if it is necessary to ship the sand from a distance. The cost of gravel and broken stone will vary in the same way as sand. If gravel is available it can usually be obtained for $1.50 to $2.00 per cu. yd. Pit run gravel commonly contains an excess of sand and fine material, and requires screening if the aggregates are to be properly graded. The cost of crushed stone is commonly greater than the cost of gravel.

**Cost of Lumber.**—The cost of lumber for falsework and forms should be obtained locally.

**Cost of Steel Reinforcement.**—The price of steel reinforcement will be the mill price plus the cost for rail and team transportation. The present (1919) base price for steel rods at the mill is about $2.50 per 100 pounds. Base prices cover bars ¾ to 3 in. in diameter. Smaller and larger bars take a higher rate as shown in Table IV. The price from stock will be from 50 cts. to $1.00 in advance of mill prices.

**Cost of Mixing and Placing.**—The labor in mixing and placing concrete will depend upon the amount of concrete and the local conditions. With a 2-bag mixer where the sand and broken stone are handled in wheel barrows the cost of mixing and placing concrete in highway bridges, with labor at an average of 40 cts. per hour including superintendence but not including profit or cost of plant, should be from 90 cts. to $1.00 per cu. yd. The allowance to be made for cost of plant will ordinarily vary from 50 cts. to 75 cts. per cu. yd. With a 4-bag mixer the cost of mixing and placing will be reduced, but the allowance to be made for plant will be increased. The cost of plant should be carefully estimated for each job, for the reason that a considerable part of the expense is independent of the size of the job.

**Cost of Forms and Falsework.**—The amount of lumber required will depend upon the actual surface of concrete and also upon the amount of lumber that can be used more than once. With a highway bridge of several spans the form lumber can be used more than once, the number of times depending upon the details of design and the details of construction. The forms and falsework should be as carefully designed as the structure which is to be built. The bill of lumber required for the falsework and forms should be taken from the plans, and the cost calculated for the local conditions. The salvage value of the falsework and forms should be deducted from the final cost. The cost of lumber should be obtained locally on small jobs.

The cost of falsework piles should be obtained locally where piles are available.

The cost of driving piles for highway bridges will vary from 40 cts. to 60 cts. per lineal foot of pile.

With carpenters at 80 cts. per hour and common labor at 40 cts. per hour, the cost of framing and placing falsework will vary from $8.00 to $12.00 per M. The cost of erecting forms will vary from $12.00 to $20.00 per M.

**Cost of Placing Reinforcement.**—The cost of bending and placing steel reinforcement will depend upon the size of the reinforcing steel and upon the skill of the men employed in doing the work. With labor at 40 cts. per hour the cost of bending and placing steel reinforcement, in highway bridges, including superintendence but not including profit, will vary from $10.00 to $15.00 per ton.

**Examples of Cost of Concrete Highway Bridges.** The following summaries of costs of constructing concrete highway bridges will be of value.

**Cost of Kearney, Nebraska, Concrete Arch Bridge.***—This bridge consisted of 14 concrete arches varying by 5 ft. increments from 55 ft. spans at the ends to 85 ft. spans at the center. The foundations extend 3 ft. below low water and rest on 40 ft. cypress piles extending 12 in. into the footings. A 1 : 2 : 4 concrete was used. A steam hammer was used in driving the piles. Concrete was mixed with a ½ cu. yd. mixer driven by steam power. Common labor was paid an average of 30 cts. per hour, carpenters 45 cts. per hour, foremen 45 to 50 cts. per hour. The quantities of materials in the bridge were, concrete 2,300 cu. yds.; reinforcing steel 59 tons; falsework piles 68 bents of four 16-ft. piles, 12 ft. penetration, or 272 piles; foundation piles 415 piles, each 40 ft. long; form lumber 131,453 ft. B.M. The cost of driving falsework piles was 49 cts. per lineal foot. The cost of driving foundation piles was 32 cts. per lineal foot. Lumber cost $36.11 per M. About one-third of the lumber was salvaged at $15.44 per M. The cost of placing steel reinforcement was $11.40 per ton. The cement cost $2.18 per bbl. at the site. The amount of cement used per cu. yd. of concrete was 1.35 bbl. Sand and gravel were obtained at the site. The contractor received 17 cts. per cu. yd. for screening sand, and 40 cts. per cu. yd. for screening gravel.

SUMMARY OF COST OF CONCRETE.

| | Foundations. 823 Cu. Yd. Per Cu. Yd. | Arches. 931 Cu. Yd. Per Cu. Yd. | Spandrel. 270 Cu. Yd. Per Cu. Yd. | Rail. 276 Cu. Yd. Per Cu. Yd. |
|---|---|---|---|---|
| Lumber, etc. | $2.65 | $2.24 | $ 2.24 | $0.65 |
| Forms, Labor | 1.36 | 1.39 | 5.19 | 3.84 |
| Cement | 2.95 | 2.95 | 2.95 | 2.95 |
| Sand and Gravel | .47 | .47 | .47 | .47 |
| Mixing and Placing | 1.31 | 1.39 | 2.15 | 1.84 |
| Total cost per cu. yd. | $8.74 | $8.44 | $13.00 | $9.75 |

**Cost of Concrete in Culverts.**†—The cost of building sixteen concrete culvert bridges in Iowa in 1918, with spans varying from 6 to 60 ft. varied from $9.49 to $23.54 per cu. yd., with an average of $12.12 per cu. yd. Three-fourths of the concrete was plain, 1–3–5 mix, and one-fourth 1–2–4 mix, with reinforcing.

* Engineering and Contracting, February 27, 1918.

† Engineering and Contracting, June 26, 1918.

**Cost of Concrete Culvert Pipe.***—At the 5th annual convention of the Iowa Road Officials at Ames, Ia., in 1918, the following data on the cost of making concrete culvert pipe at the plant of Adair County were given by C. H. Lehmkuhl, engineer of the county:

| Size of Pipe, In. | Thickness, In. | Length, Ft. | Concrete in Each Pipe, Cu. Ft. | Cost per Pipe. | | |
|---|---|---|---|---|---|---|
| | | | | 1915. | 1916. | 1917. |
| 15 | 2¼ | 3 | 3.00 | $1.74 | $1.80 | $1.95 |
| 18 | 2½ | 4 | 4.50 | 2.72 | 2.80 | 3.00 |
| 24 | 3 | 4 | 6.62 | 4.08 | 4.20 | 4.40 |
| 30 | 4 | 6 | 17.80 | 8.04 | 8.40 | 8.76 |
| 36 | 4 | 6 | 20.90 | 9.84 | 9.90 | 10.56 |

* Engineering and Contracting, June 12, 1918.

# CHAPTER XXVI.

## Erection of Bridges.

**Introduction.**—The construction of concrete highway bridges and foundations, and the erection of steel highway bridges will be briefly discussed in this chapter. For additional data on the erection of steel bridges and for details of erection tools and equipment, see the author's "Structural Engineers' Handbook."

**CONSTRUCTION OF CONCRETE BRIDGES.**—The details of mixing and placing concrete are given in the "General Specifications for Concrete Highway Bridges and Foundations" in Appendix II. A brief summary of the important points will be given here. Good clean sand and stone or gravel are absolutely essential. While a small amount of pure clay may not be detrimental it is rare that pure clay occurs in sand and stone or gravel; the impurities more often being loam and other deleterious matter. While a small amount of crusher dust properly mixed may not be objectionable, it is objectionable when it clings to the broken stone and prevents proper bonding of the mortar. The fine and coarse aggregates must be in the proper proportion and be properly graded from fine to coarse. Gravel very rarely occurs in the proper proportions of fine and coarse aggregate to be used without screening. Proper proportions of cement, fine and coarse aggregate require that there shall be slightly more cement than is necessary to fill the voids in the sand, and slightly more mortar than is necessary to fill the voids in the stone. It is also necessary that the fine and coarse aggregates be so proportioned as to give a concrete of maximum density with the given quantity of cement. The relative amount of fine and coarse aggregates should be determined either by mechanical analysis or by means of trial proportions. For determining the proper proportions by trial a pair of scales and several measuring vessels are all the apparatus necessary. By varying the proportions and mixing the ingredients the densest mixture may be determined by weighing. The percentage of voids in fine and coarse aggregate may be determined by weighing a measured quantity of aggregate. The specific gravity of quartz sand is closely 2.65. For example, a sand weighing 100 lb. per cubic foot will have solid contents equal to $100/(2.65 \times 62.5) = 60.4$ per cent; and will have 39.6 per cent voids. The aggregate should be dried before determining the percentage of voids. If the percentage of voids in fine and coarse aggregates are determined separately, the voids should also be determined for the fine and coarse aggregates when completely mixed. In one case coming to the author's attention in investigating a failure of a reinforced concrete flume, while the percentage of voids in the sand and gravel when determined separately was each 35 per cent, the percentage of voids in the sand and gravel when mixed in the proportion of 2 sand to 5 gravel was 33 per cent, due to the fact that the sand was coarse and the gravel contained a large percentage of pea gravel. In a 1–2–5 mixture the cement paste filled only about 40 per cent of the voids. The resulting concrete was a weak chalky mass which fell to pieces when water was turned into the flume. A field test of this concrete, made by breaking a concrete specimen with a hammer, would have shown the quality of the concrete and saved the structure.

Concrete should be thoroughly mixed with only sufficient water to make a pasty mass, not a soupy mass, after vigorous mixing in a mixer that will thoroughly grind the mortar and fully coat the fine and coarse aggregates. Soupy concrete results in pockets, and porous concrete, and sometimes distinct lines of separation through a wall or a girder. The proper consistency is the stiffest mixture that will admit of proper contact with the reinforcement and with the forms. To secure this contact the concrete should be puddled and joggled and tamped to remove all air pockets

and to fill all voids. Concrete deposited under water through tremies should be thoroughly mixed before depositing. Properly mixed concrete making a stiff mixture will not separate as much as dry concrete when deposited under water.

**Forms.**—Forms should be rigid and should be carefully finished. The forms should be designed to support the loads, and every precaution should be taken to prevent sagging and leakage during construction. The corners should be properly filleted and the details properly worked out to give a pleasing appearance to the finished structure. For all important work the lumber used for face work should be dressed to uniform thickness and width, should be sound and free from loose knots, and should be secured to the studding or uprights in horizontal lines. For backing and other rough work undressed lumber may be used. Lumber used a second time should be cleaned, and resized if necessary to insure plane surfaces.

**Design of Falsework and Forms.**—Falsework and forms should be designed to prevent undue deflection, and to prevent the crushing of the timber across the grain. Where falsework and form lumber is carefully selected the allowable stresses given for timber in "Specifications for Timber Bridges and Trestles" in Chapter XVI may be increased by twenty-five per cent. In designing falsework and forms it should always be remembered that rigidity is more important than low fiber stresses. The spans should therefore be kept small and ample supports and ties should always be provided.

The practice of the Illinois Highway Commission in designing forms for reinforced concrete girder bridges is as follows:

"1. *Rail and Girder Forms.*—Forms for side rails of reinforced concrete slab bridges may be constructed of 1-inch sheeting with vertical studs placed not farther apart than 2 feet. Forms for the girders of reinforced concrete girder bridges should preferably be constructed of commercial 2-inch sheeting with studs not farther apart than 2½ feet. All sheeting should be surfaced on the side adjacent to the concrete.

"2. *Bracing Rail Forms.*—Rail or girder forms are best kept in line by extending the caps of each bent a sufficient distance, bracing them to the falsework posts and then running a heavy string-piece along the ends of the extended caps, bracing from this string-piece to each stud of the rail or girder forms.

"3. *Setting Panel and Coping Forms.*—The panels and coping of rail and girder forms should, whenever practicable, be omitted until the floor of the span has been concreted. The weight of the floor is usually the greater part of the total weight of the superstructure and if any settlement of the falsework occurs, it is usually when the floor is placed. If the panel and coping forms are completed before any concrete is placed, settlement of the falsework will show in the panels and coping. It is not safe to trust to a camber to take care of the settlement, as the settlement is almost sure to be uneven at the different supports. The side forms of the rails or girders should preferably be left 3 or 4 inches higher than the finished girder, and just before the last concrete is placed, a triangular molding should be nailed on the inside of the forms at the exact elevation required and used as a guide for a template in striking off the top of the girder. If these precautions are taken the portions of the work visible from the roadway may show perfect lines, although a settlement of the concrete may have occurred which shows as a sag in the bottom of the girders when viewed from the side. A small settlement of falsework which occurs before the concrete has set does not injure the strength of the bridge.

"4. *Construction of Girder Forms.*—Girder forms should be so built as to permit of ready removal without injury to the concrete. The underside of copings should be given a pitch towards the girder for this purpose. Great care should be taken to secure perfect alignment of rail and girder forms. Local kinks should be taken out before the concrete is placed.

"5. *Alignment of Forms.*—Correct alignment of girder and rail forms can not be too strongly emphasized. Irregular lines are exceedingly unsightly and as the bridge will be judged for all time to come from the appearance of the portion visible from the roadway, if this appearance is unsightly, the bridge will be condemned by the public regardless of the possible excellence of the concrete."

The following instructions for constructing concrete highway bridges were prepared by Mr. M. W. Torkelson, bridge engineer, Wisconsin Highway Commission.

"1. Never place the bents for any kind of falsework more than five feet apart.

"2. If you can possibly get a pile driver, use driven piling to support your falsework.

"3. If you cannot get a pile driver, good bottom can be obtained by laying planks or timbers on the stream bed to get a good wide footing for each bent, then placing the mud sill upon the

planks to support the posts. Before the planks or timbers are laid upon the stream bed this should be leveled and all soft mud or easily moved sand should be removed. Sometimes temporary concrete sills can be used to advantage, but the principle is the same as for wooden sills.

"4. The posts for the bents should be eight inches thick and of good sound timber. Always use eight posts in each bent where the road is 20 ft. wide, and arrange the posts so that two will come under each railing.

"5. Use a heavy cap on top of the posts fastening either with dowels or by means of very heavy spiking, and let this cap extend about four feet beyond the railing.

"6. The floor can best be directly supported by $2 \times 10$ joists spaced 18 inches. Under railings double this up.

"7. With the joists spaced as in (6) the floor can be 1-inch or $\frac{7}{8}$-inch material. It should be nailed to the joists with 8-penny nails and should extend about four feet beyond the outside of the railing. This extra width is needed to brace the railing and for walking across the bridge.

"8. Be sure to cross-brace the falsework both ways so that it will be held rigid against pressure from any direction. Unless this is looked after the falsework is liable to wobble when the placing of the concrete begins.

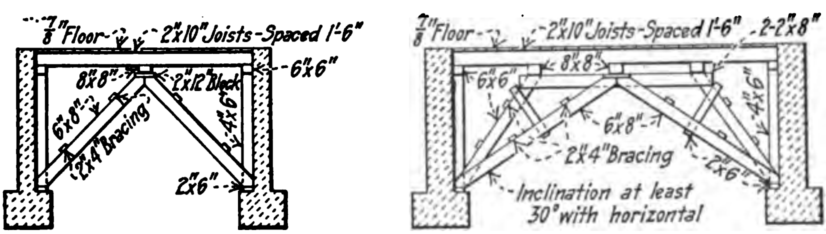
"9. Always use good planed and sized lumber on the railing as this is the part of the work which shows up. Have all panel work and three cornered chamfer strips made at the same planing mill. Send your bridge plan to the mill and have strips cut to the proper dimensions. Wet railing forms thoroughly before pouring concrete.

"10. Always keep a man tamping the concrete next to the forms. This will give a good smooth surface when they are removed and diminish the work of finish. A wooden tamper will give better results than a steel spade.

"11. *Removal of Forms.* In order to make possible the obtaining of a satisfactory surface finish, forms, on ornamental work, railings, parapets, and vertical surfaces that do not carry loads and which will be exposed in the finished work shall be removed in not less than twelve (12) nor more than forty-eight (48) hours, depending upon weather conditions. Forms under slabs, beams, girders, and arches shall remain in place at least twenty-one (21) days in warm weather, and in cold weather at the discretion of the Engineer. Forms shall always be removed from columns before removing shoring from beneath beams and girders, in order to determine the conditions of column concrete.

"No forms whatever shall be removed at any time without the consent of the Engineer. Such consent shall not relieve the contractor of responsibility for the safety of the work. As soon as the forms are removed all rough places, holes, and porous spots shall be filled, and all bolts, wires, or other appliances used to hold the forms and which pass through the concrete shall be cut off or pushed back with nail set one-half ($\frac{1}{2}$) inch below the surface and the ends covered with cement mortar of the same mix as used in the body of the work."

The falsework and form plans given in Fig. 1 to Fig. 4 were prepared by Mr. M. W. Torkelson, bridge engineer for the Wisconsin Highway Commission.




(a) (b)

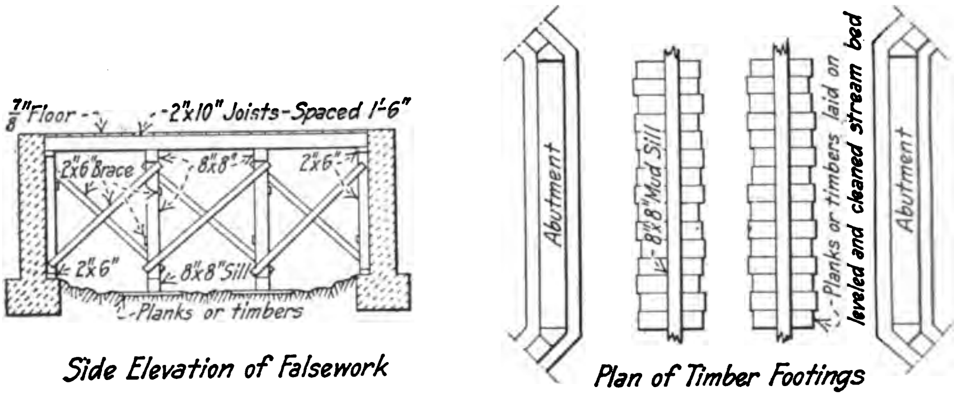
FIG. 1. FALSEWORK FOR CONCRETE BRIDGES.

The plans shown in (*a*) Fig. 1, are for spans up to and including 14 ft.; while the plans shown in (*b*) Fig. 1, are for spans of 16 ft. to 24 ft. inclusive. The bents should be spaced 5 ft. centers. Use four 2 in. by 10 in. joists under the railing. Use hardwood wedges for camber and to facilitate removal of forms.

The falsework in Fig. 2 should be used where it is impossible to drive piles. Bents should be spaced not more than 5 ft. centers and two posts should be spaced under the railing. Use four 2 in. by 10 in. joists under the railing. Use 2 in. by 6 in. joists on top of footing under each joist.

Use hardwood wedges for camber and to facilitate erection. Details of an elevation of a bent are shown in Fig. 3.

The falsework bent shown in Fig. 3 may be a framed bent supported on a mudsill as is shown on the left, or may be a pile bent as is shown on the right. Eight posts or eight piles should be used in a bent for a 20 ft. roadway. Two posts or piles should be spaced close together under the




(a) (b)

FIG. 2. FALSEWORK FOR CONCRETE BRIDGES.

railing. Bents should be spaced not more than 5 ft. centers. Use four 2 in. by 10 in. joists under the railing. Use hardwood wedges for camber and to facilitate erection. Pour floor to top of curb, then build inside railing form to top of 2 in. by 6 in. plate, or under side of coping, and pour rail to this point. Let concrete set while pouring other rail to this height. Return to first rail and see if any settlement has occurred, and if so wedge up 2 in. by 6 in. plate level before building form for coping. Always provide camber in forms ½ in. for each 10 ft. of span. Have three-cornered strip made at planing mill. For elevation and section of railing forms see Fig. 4.

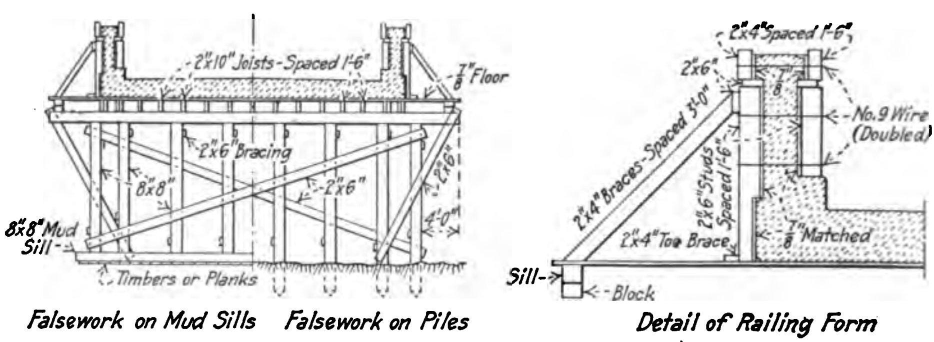



FIG. 3. FALSEWORK PLANS AND RAILING FORMS FOR CONCRETE BRIDGES.

Details of forms for retaining walls as constructed by the Illinois Central R. R. are shown in Fig. 5. The forms were constructed in sections 54 ft. long. The forms were cross-braced by ¾-in. rods spaced 7 ft. 8½ in. centers as shown. When the forms were taken down the ends of these rods were unscrewed, the main portions of the rod being left in the wall. The forms were made of 2-in. plank surfaced on the inside.

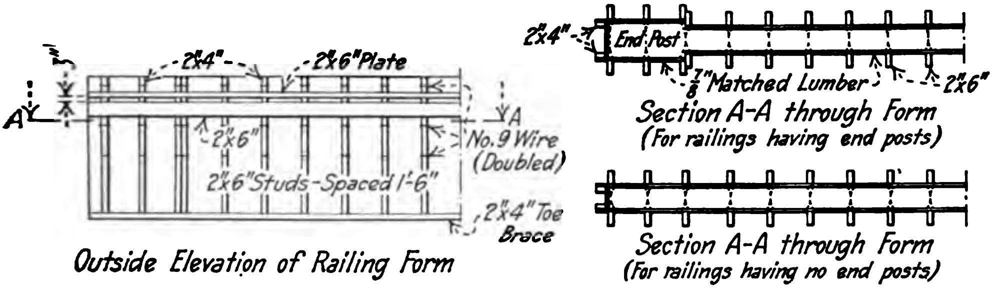



FIG. 4. FORMS FOR RAILINGS FOR CONCRETE BRIDGES.

Forms used by the Chicago and Northwestern Ry. are shown in Fig. 6. The forms were built in sections 35 ft. long. The 2 in. × 8 in. braces were used to hold the sides of the forms apart and were removed as the concrete was put in place. The 2-in. pipe used to cover the rod bracing was old boiler flues and rejected pipe.

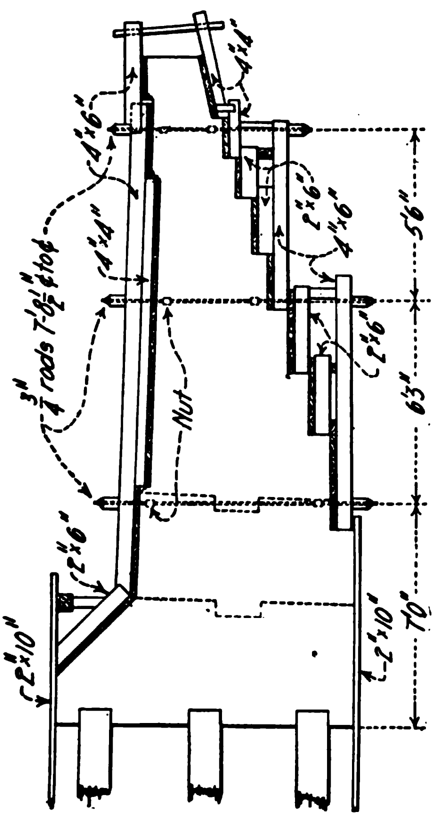



FIG. 5. FORMS FOR ILLINOIS CENTRAL R. R. RETAINING WALL.

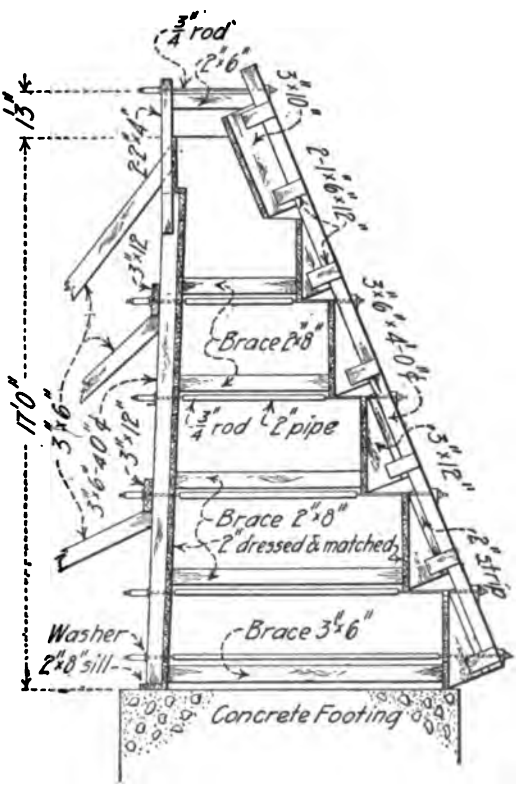



FIG. 6. FORMS FOR C. & N. W. RY. RETAINING WALL.

The forms for concrete arch culverts as prepared by the Michigan State Highway Department are shown in Fig. 7.

**Falsework for Arches.**—The detail plans for the falsework used in the erection of a concrete arch bridge on the joint track of the Colorado and Southern Ry. and the Denver and Rio Grande R. R. in Colorado are shown in Fig. 8. The bridge consisted of twin arches each having a span of 60 ft., and a barrel 112 ft. long. The falsework was designed to carry the actual loads which would come on the falsework during erection. The falsework was made very rigid in order that there should be no appreciable settlement. The falsework was constructed with a barrel having a length of 60 ft., so that one-half of each arch could be constructed at one time. The abutments

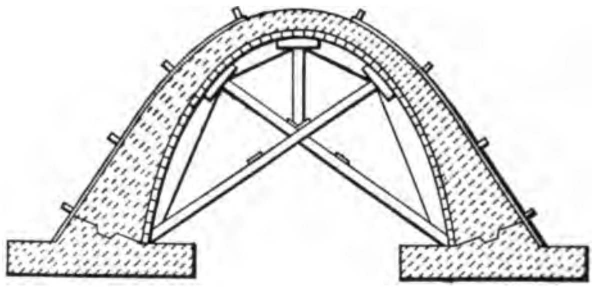



FIG. 7. FORMS FOR CONCRETE ARCH CULVERT.

were first constructed and then the falsework was constructed for one-half of each arch. The concrete was then placed on each arch beginning at the springing and proceeding upward to the crown. The crown segments were constructed last. The arches were not reinforced, except that a small amount of reinforcing steel was placed near the extrados of each arch to make it possible for the ends of each arch to act as a cantilever until the crown segment was placed. The falsework proved to be very rigid, the maximum settlement noted in the arch sheeting was one hundredth of a foot, with no appreciable distortion. The forms were lowered by means of the sand boxes shown in the drawing. After several years the arches show no cracks.

After the one-half of the bridge was constructed the falsework was taken down and re-erected for the remaining half of the bridge. The additional cost required to make the falsework very rigid was more than compensated for by the saving in cost of placing the concrete. The arch was designed and constructed by Crocker and Ketchum, consulting engineers. The author was in direct charge of the design of the arch and the falsework.

**Lagging.**—Lagging for concrete arches should be of surfaced lumber, preferably tongue and groove, and should be water tight.

**Construction of Concrete Arches.**—The arch ring should not be constructed until the fill around the abutments has been carried up to the skewback. The rings should preferably be concreted in one continuous operation, but if this is not practicable the arch ring may be divided into several sections by transverse bulkheads parallel to the roadway, each ring being of such size that it can be concreted in a single continuous operation. The concreting should be carried on symmetrically about the crown of the arch. If the arch ring is heavy additional reinforcement should be inserted near the extrados over the haunch so that the segments of the arch will act as cantilevers until the arch is closed at the crown. The spandrel walls should not be cast until the centers are struck, and the coping should not be cast until the spandrel wall is completed. On very large arches it may be necessary to divide the arch ring into voussoirs, so that the arch ring can be poured in such a manner as to load the centers symmetrically. The

extrados of the arch ring and the inside surface of spandrel walls snould be left smooth to receive the waterproofing. The surface may be waterproofed as described in § 75, Appendix II, or the membrane method may be used. Before applying the membrane the surface of the concrete should be clean and dry and not less than 15 days old. A primer coat should be applied cold. For asphalt the primer coat should be asphalt thinned with petroleum distillate; while for coal tar the primer coat should be creosote oil which shall be a pure tar distillate free from any sub stance foreign to a tar distillate. The membrane should be applied, so as to lap joints as for tar and gravel roofs. The surface of the concrete and of all laps are to be mopped with hot asphalt or hot tar. Especial care must be used to flash the concrete in angles and to provide the necessary expansion joints. For detai's of waterproofing concrete bridge floors, see the author's "Structural Engineers' Handbook."

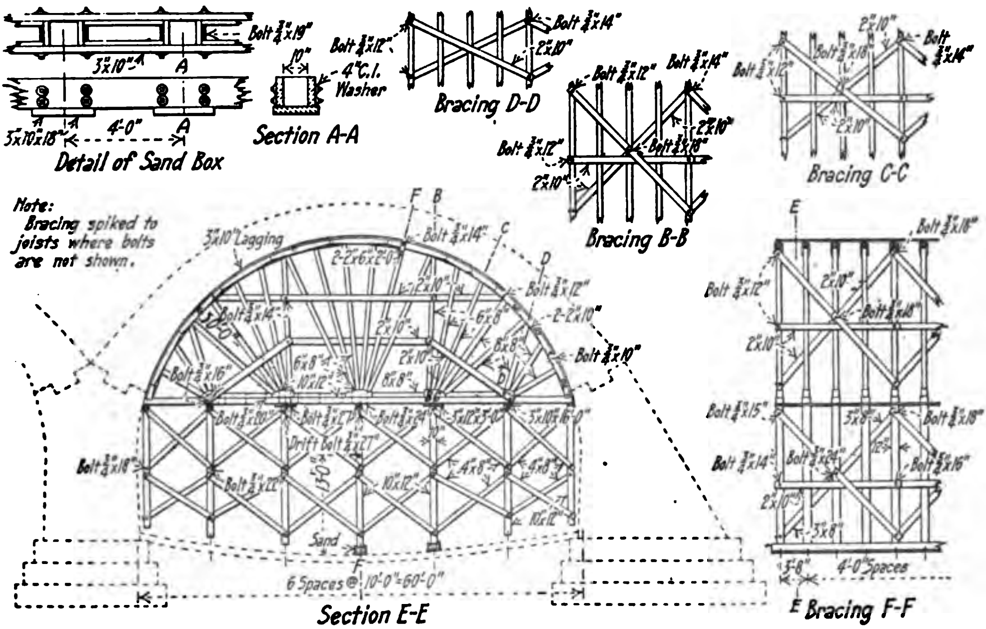



FIG. 8. FALSEWORK FOR CONCRETE ARCH BRIDGE, C. & S. RY. AND D. & R. G. R. R.

Spandrel filled arches should be drained by French drains 15 in. square provided with suitable tile outlets. Drains should be provided for all abutments and retaining walls. Filling of spandrel filled arches should be deposited in layers 6 in. to 8 in. thick, and thoroughly compacted by ramming. The fill should be made symmetrically from both ends of the arch.

**Striking Centers.**—Centers should be gradually and uniformly lowered in such a manner as not to produce injurious stresses. The forms for small span arches should be supported on hard wood wedges, while sand boxes should be used for large span arches. In mild weather, centers should remain in place under arches of less than 60 ft. span for at least 21 days, and under arches over 60 ft. span for at least 28 days.

**Depositing Concrete Under Water.**—The depositing of concrete under water should be avoided if possible, as results are somewhat uncertain even when the work is done under strict supervision. The methods that give the best results are:

1. The concrete is lowered in large buckets having a hinged bottom which sets sufficiently far above the lower edge of the bucket that it may open freely downward and outward when the

bucket reaches the surface upon which the concrete is to be deposited. The top of the bucket is left open. The bucket should be completely filled before lowering.

2. The concrete is deposited through a vertical tube or "tremie" reaching down to the surface upon which the concrete is to be deposited. The tremie should be kept filled and the flow of concrete should be continuous. In beginning the operation the tremie should be filled with concrete in such a manner that the concrete is not permitted to drop through the water. This may be accomplished by plugging the tremie with sacks which will be forced down as the tremie is filled with concrete, by plugging the end of the tremie with a cloth sack filled with cement, or by other means. If the charge is lost the tremie should be filled before proceeding.

3. The concrete may be deposited in loosely filled porous cloth or jute bags. These bags are placed so as to bond together. The mortar working through the porous bags cements the mass.

4. Premoulded concrete blocks of large dimensions may be used.

5. A canvas bag may be used as a depositing bag in place of a bucket. After filling, the mouth of the bag is closed by one turn of a line so looped that a pull on the line will release it. The bag is lowered mouth down to the surface upon which the concrete is to be deposited, and a pull on the line opens the bag and releases the concrete.

The following precautions should be taken in depositing concrete under water:

(*a*) The concrete should be made with aggregate smaller in size than for concrete deposited in air. The aggregate should be carefully graded so as to make a dense mixture. The mix should be not less than 1–2–4 mix, and should contain more cement than for concrete deposited in air. The concrete should be thoroughly mixed in a batch mixer with only sufficient water to make a stiff mass. Concrete should never be deposited in running water. In running water a cofferdam should be constructed in such a manner as to insure still water within the cofferdam. The concrete shall be deposited continuously in order that laitance may not form between the layers of concrete.

(*b*) Before beginning concreting after an interruption the laitance should be removed from the surface of the concrete already placed. It is impossible to prevent the formation of laitance, but great care should be taken to reduce the amount of laitance and also to prevent the formation of horizontal cracks. Concrete should not be deposited in water the temperature of which is cold enough to retard setting.

**Placing Reinforcement.**—The vertical steel in abutments and piers should be in place and be rigidly supported before concreting is started. The horizontal steel should be wired in place in advance of the concrete as indicated on the plans. All the steel in the superstructure should be wired in place before any concrete is deposited in the forms. Great care should be used to see that the steel is located exactly as shown on the drawings. Reinforcing steel should be supported on metal or other approved supports to hold it at the proper distance above the forms. The practice of laying reinforcing steel directly on the forms and attempting to raise the steel during construction is pernicious and should not be permitted.

**Inspection of Design and Construction of Concrete Structures.**—The construction of concrete structures should not be separated from the design, but the engineer who prepares the design should supervise the construction.

The design drawings and specifications should give the dead, live and wind loads, the allowance for impact, the working stresses, and the arrangement of all details. The drawings should show the size, length, location of points of bending, and exact position of all reinforcement, including stirrups, ties, hooping and splicing. The specifications should state the qualities of all materials and the proportions that are to be used.

Plans should also be prepared by the engineer for all falsework and forms. Alternate plans for falsework and forms should be invited from experienced contractors.

Inspection during construction should be made by the engineer's inspectors, and should cover the following:

1. Tests and inspection of materials. 2. Construction and erection of falsework and forms. 3. Sizes, arrangement, position and fastening of reinforcement. 4. Proportioning, mixing, consistency, and placing of concrete. 5. Tests of concrete made on work. 6. Testing concrete to see if it is sufficiently hardened before supports are removed. 7. Protection of finished parts of structure from injury. 8. Comparison of dimensions of all finished parts of structure with plans. 9. Inspection of finish of concrete.

**ERECTION OF STEEL HIGHWAY BRIDGES.**—The details of the operation of erecting steel highway bridges will depend upon the type of bridge, length of span and character of the crossing. Short span plate girder and riveted truss bridges may be riveted or bolted up on the bank, and then swung in place by means of a gin pole (a long pole held solidly at the bottom and held in place at the top by guy ropes; the load is lifted by blocks and falls fastened to the top and bottom of the pole, while the load is swung into place by manipulating the guy ropes). Pin-connected bridges of all spans and long span riveted truss bridges are erected on falsework, usually constructed of timber.

Through truss bridges are usually erected by means of a gantry overhead traveler which runs on a track supported on the falsework. Details of a through bridge traveler are shown in Fig. 9. The falsework may be made of framed bents as shown in Fig. 10, or pile bents may be used.

**Falsework.**—Falsework for the erection of bridges is built up of bents made of three or more posts or piles, braced transversely in the same manner as for permanent trestles. Framed bents are carried on mudsills, or on piles when the foundation is inadequate or where there is flowing water. Where piles cannot be driven in running water or where there is danger of flood, it may be necessary to use spread footings which are anchored in place. Where it is practicable to obtain piles of sufficient length they may be used for the full height of the falsework. The timber used in building falsework should be sound, strong, free from defects that will affect its strength or interfere with its use. Since the structure is temporary, durability is not an important element in selecting timber for falsework unless it is to be used several times.

For examples of timber trestles, see Chapter XVI.

Plans of typical four-legged falsework as used by the American Bridge Company are shown in Fig. 10. When trains are to be carried and 2–8 in. × 16 in. stringers are used under each rail, bents must not be spaced over 18 ft. centers for the falsework as shown.

**Piles.**—Timber piles may be driven with a drop hammer or with a steam hammer. A spool roller pile driver with a drop hammer is shown in Fig. 11. The hammer is raised to the top of the leads by the hoisting engine; the hammer is then permitted to fall on the top of the pile, dragging the hoisting rope down with it. The force of the blow of the hammer depends upon the weight of the hammer, the height of free fall, and the resistance of the hammer in the leads. By catching the hammer as it descends the operator can cushion the blow so that the safe bearing power of a pile as calculated from the penetration may be very misleading.

The safe load on piles may be calculated by the Engineering News formula given in § 82 of the "General Specifications for Concrete Highway Bridges and Foundations," Appendix II. Piles should have a penetration of not less than 10 ft. in hard material and not less than 15 ft. in soft material.

The following specifications may be used for falsework piles. All piles are to be spruce, yellow pine or oak, not less than 8 in. in diameter at the tip and not more than 14 in. in diameter at the butt. Piles are to be straight and sound, and free from defects affecting their strength. Piles are to be driven into hard ground until they do not move more than ½ in. under the blow of a hammer weighing 2,000 lb. and falling 25 ft.

**Erection of a Through Truss Bridge.**—The following description of the erection of a 409-ft. Petit through pin-connected highway bridge will illustrate the method of erecting truss bridges. The falsework was constructed by driving 5 lines of piles to a good refusal. The piles were sawed off and capped with 12″ × 12″ timbers. The longitudinal sills for the traveler were 10″ × 12″

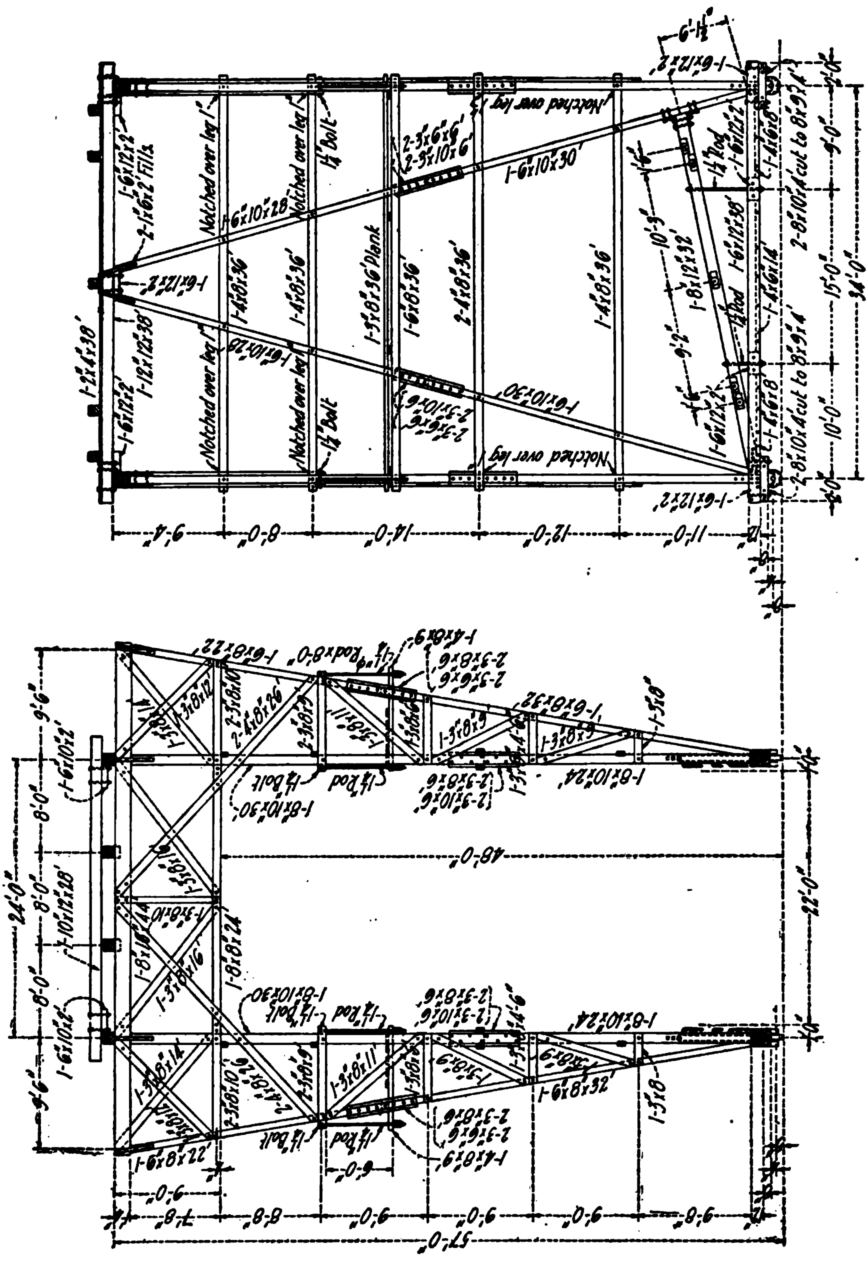


FIG. 9. DETAILS OF THROUGH BRIDGE TRAVELER. AMERICAN BRIDGE COMPANY.

timbers, while 3″ × 10″ timbers were used for transverse and longitudinal bracing. The pile bents were spaced 29 ft. centers, which was the same as the panel length of the bridge. The traveler was built so that it would clear the bridge when it was erected, and was 58 ft. long. After the falsework was completed the traveler was erected and moved out on the track to the center of the bridge. The floorbeams and lower chord bars were then put in place on the falsework, care being used to see that the pedestals would come in their proper places.

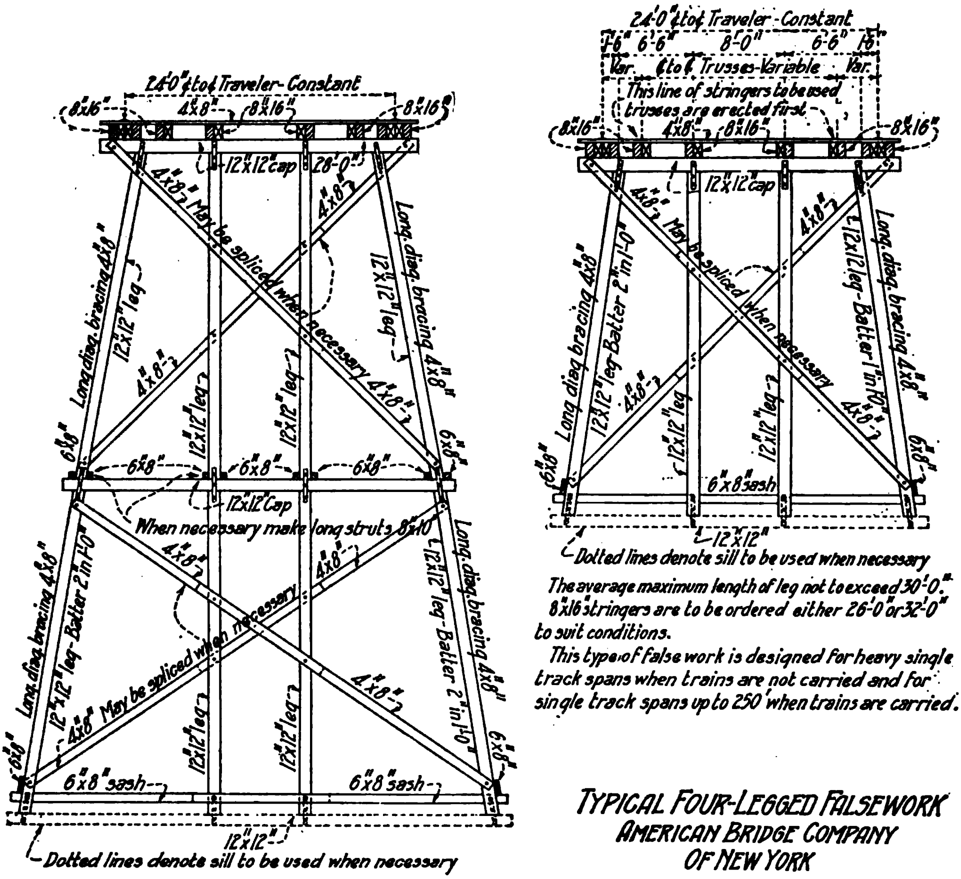



FIG. 10.

The four vertical posts forming the middle double panels were then lifted into place by means of a hoisting engine and were bolted to the floorbeams. The diagonals were then put in place and the posts, the diagonals and the lower chords were connected up. The middle sections of the top chord were then put in place and the diagonals, top struts, sway and lateral bracing were put in place. The middle panel of the bridge was now self-supporting. The traveler was then moved 58 ft. toward one end of the bridge and the next double panel was erected, and so on until finally the end-posts were erected. The traveler was then taken back past the center of the bridge and the other end was erected in the same manner. The blocking was then knocked from under the panel points and the span was swung free. The riveting was then completed, the floor joists and floor covering were put in place and the bridge was painted.

Pilot points and driving nuts, as shown in Fig. 17, Chapter XV., are used in driving chord pins to protect the threads.

In erecting deck bridges the traveler is often run on the completed part of the span. Steel trestles may be erected from a traveler run on top of the completed structure; or the bents may be riveted up on the ground and then erected by using a gin pole, and after the towers have been erected the girders are raised in place by means of gin poles fastened to the tops of the towers.

In erecting small highway bridges of, say, 100-ft. span, a traveler is not ordinarily used. After building the falsework, as previously described, the four vertical posts near the center, together with the middle sections of the top chord, are raised by means of gin poles, a hand crab being used in place of a hoisting engine.

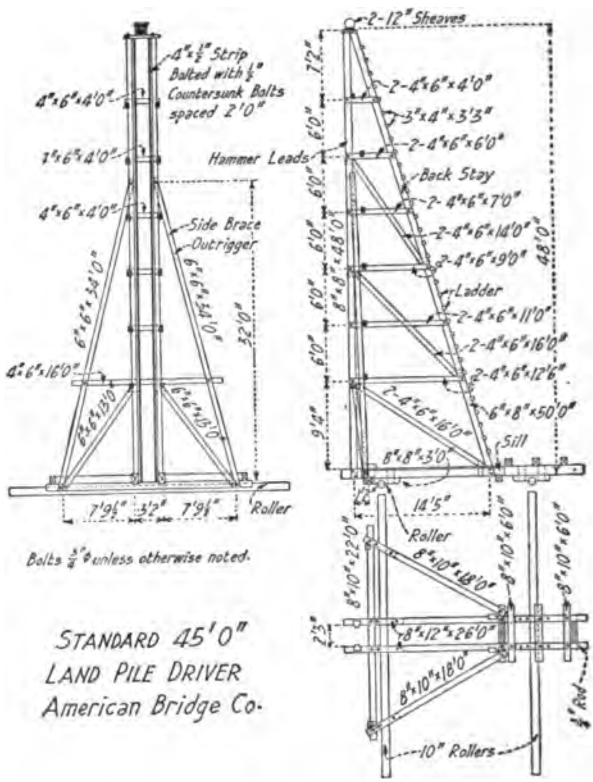



FIG. 11. DETAILS OF STANDARD PILE DRIVER. AMERICAN BRIDGE COMPANY.

The top chord of bridges should be designed with special reference to the methods used in erecting the bridge. In bridges with parallel chords, the middle section of the top chord should be detailed so that the middle panel of both chords may be erected and made self-supporting. Splices in top chords should be placed as near panel points as practical, and between the panel point and the nearest end of the bridge. In bridges with inclined chords no splices are required, the stresses in the chords being transferred directly through the pin.

**Erection Equipment.**—Details and description of erection equipment, including derricks, hoists, and erection tools are given in the author's "Structural Engineers' Handbook."

**Mill Inspection.**—The details of mill inspection are given in "General Specifications for Steel Highway Bridges" in Appendix I. While the product of rolling mills is quite uniform, mill inspection of the material is very desirable. The mill inspection is ordinarily done by a representative of a testing concern who is stationed at the mill. Where material is taken from stock it is necessary to waive mill inspection.

**Shop Inspection.**—The drawings are checked, and the fabricated members are checked with the drawings. All defects discovered are corrected. All defective material is replaced. The following items are important in shop inspection. Note that the various pieces of which a member is to be constructed are straight and free from dust or dirt. Surfaces coming in contact should be painted two coats of paint before being riveted together. Rivet holes should match so that hot rivets may enter the hole without driving. Drift pins may be used for drawing the parts of the member together, but not for enlarging rivet holes. If holes do not match they should be reamed. The pieces should be tightly drawn together with bolts before the rivets are driven. For usual conditions about twenty-five per cent of the holes should be filled with erection bolts, tightly drawn up before any rivets are driven. If the plates are not well drawn up the hot metal will flow between the plates, resulting in defective rivets and a loose connection. Rivets should be driven with a direct acting power riveter. Rivets driven by hand or with a pneumatic hammer should be very carefully inspected. The heads should be full and concentric. If insufficient stock is used the head will be formed without filling the hole or the heads will not be fully formed, and the rivets will work loose. With too much stock a lip will be formed around the head. Stock that will give a small amount of lip is much better than scant stock. Pinholes should be bored after the member is completely riveted. The holes should be at right angles to the axis of the member, and should be of the proper size and exactly spaced. Reamed holes are first punched to a smaller diameter and are then reamed with twist drills to the required size. Holes should be reamed with the members as in final position or a steel template should be used. The thickness and dimensions, of all plates and the thickness and weights of all structural shapes should be checked.

All rivets in the finished members should be tested for tightness by striking the head of the rivet a sharp blow with a light hammer. Loose rivets give a dull, hollow sound. If the finger is held on the side of the head on the opposite side when the blow is struck, a loose rivet can easily be detected. Great care must be used in culling out loose rivets to prevent loosening other rivets in a connection. A single loose rivet in a joint with sufficient sound rivets may be permitted to remain, if its removal is liable to loosen other rivets in the connection. The painting should be carefully inspected to see that the paint is thoroughly mixed with first class materials, that the metal is clean, dry and warm when the paint is applied, and to see that the paint is well spread and worked in with round brushes. The first coat of paint on structural steel is the most important one. If the shop coat of paint is porous and irregular the later coats will be ineffective. The only solution will be to thoroughly scrape and clean off the old paint before applying the field coat.

**Field Inspection.**—If shop inspection has not been made the material should be inspected before it is erected in place. The connections should be field bolted with at least fifty per cent of the holes filled with drift pins and field bolts. The plates must be drawn tightly together before any rivets are driven. Before rivets are driven in the main trusses the camber blocking should be removed. Milled joints should be square and in full bearing. Make sure that the expansion rollers or rockers are properly located.

**REFERENCES.**—For additional data on the erection of bridges consult the following; Ketchum's " Structural Engineers' Handbook; Hool and Johnson's " Concrete Handbook."

# APPENDIX I.

## GENERAL SPECIFICATIONS FOR STEEL HIGHWAY BRIDGES.*

BY

MILO S. KETCHUM,

M. Am. Soc. C. E.

FOURTH EDITION,

1920.

## PART I. DESIGN.

### GENERAL DESCRIPTION.

1. **Classes.**—Bridges under these specifications are divided into eight classes, as follows:
Class A.—For city traffic.
Class B.—For suburban or interurban traffic with heavy electric cars.
Class C.—For country roads with ordinary traffic and light electric cars.
Class $D_1$.—For country roads with heavy traffic.
Class $D_2$.—For country roads with light traffic.
Class $E_1$.—For heavy electric street railways only.
Class $E_2$.—For medium electric street railways only.
Class $E_3$.—For light electric street railways only.

2. **Material.**—All parts of the structure shall be of rolled steel, except the flooring, floor joists and wheel guards, when wooden floors are used. Cast iron or cast steel may be used in the machinery of movable bridges, for wheel guards, and for bed plates and rockers.

3. **Types of Truss.**—The following types of bridges are recommended:
Spans up to 30 ft.—Rolled beams.
Spans from 30 to 80 ft.—Riveted plate girders, or riveted low trusses for classes A, B, $E_1$, $E_2$ and $E_3$; and riveted low trusses for classes C, $D_1$ and $D_2$.
Spans 80 to 160 ft.—Riveted or pin-connected high trusses.
Spans 160 to 200 ft.—Pin-connected trusses of the Pratt type with inclined chords.
Spans over 200 ft.—Pin-connected trusses of the Petit type or K-type.

4. **Length of Span.**—In calculating the stresses the length of span shall be taken as the distance between centers of end pins for pin-connected trusses, centers of end bearing plates for riveted trusses and for girders, and center to center of trusses for floorbeams.

5. **Form of Trusses.**—The form of truss shall preferably be as given in paragraph 3. In through trusses the end vertical suspenders and the two panels of the lower chord at each end shall be made rigid members if the wind load produces a reversal of stress in the lower chord. In through bridges the floorbeams shall be riveted above or below the lower chord pins.

6. **Lateral Bracing.**—All lateral and sway bracing shall preferably, and all portal bracing must be, made of shapes capable of resisting compression as well as tension, and shall have riveted connections. Low trusses and through plate girders shall be stayed by knee braces or gusset plates at each floorbeam.

7. **Spacing of Trusses.**—For bridges carrying electric cars the clear width from the center of the track shall not be less than 7 ft. at a height exceeding one foot above the track where the tracks are straight, and an equivalent distance when the tracks are curved. The distance between centers of trusses shall in no case be less than one-twentieth of the span between the centers of end-pins or shoes, and shall preferably not be less than one-twelfth of the span.

8. **Head Room.**—For classes A, B, C, $D_1$, $E_1$, $E_2$ and $E_3$ the clear head room for a width of eight (8) ft. on each track, or eight (8) ft. on the center line of the bridge shall not be less than 15 ft., and for class $D_2$ not less than 12½ ft.

9. **Footwalks.**—Where footwalks are required, they shall generally be placed outside of the trusses and be supported on longitudinal beams resting on overhanging steel brackets.

10. **Handrailing.**—A strong and suitable handrailing shall be placed at each side of the bridge and be rigidly attached to the superstructure.

11. **Trestle Towers.**—Trestle bents shall preferably be composed of two supporting columns, two bents forming a tower; each tower thus formed shall be thoroughly braced in both directions and have struts between the feet of the columns. The feet of the columns must be secured to an anchorage capable of resisting one and one-half times the specified wind forces (§89).

* To accompany "General Specifications for Construction of a Highway Bridge," Chapter XXIV, page 430.

Each tower shall have a sufficient base, longitudinally to be stable when standing alone, without other support than its anchorage. Tower spans for high trestles shall not be less than 30 ft.

12. **Proposals.**—Contractors in submitting proposals shall furnish complete stress sheets, general plans of the proposed structures, and such detail drawings as will clearly show the dimensions of all the parts, modes of construction and sectional areas.

13. **Drawings.**—Upon the acceptance and the execution of the contract, all working drawings required by the engineer shall be furnished free of cost (§168).

14. **Approval of Plans.**—No work shall be commenced or materials ordered until the working drawings have been approved by the engineer in writing.

## FLOOR SYSTEM.

15. **Floorbeams.**—All floorbeams shall be rolled or riveted steel girders, rigidly connected to the trusses at the panel points, or may be placed on the top of deck bridges at panel points. Floorbeams shall preferably be square to the trusses or girders.

16. **Joists and Stringers.**—All joists and stringers of bridges of classes A, B, $E_1$, $E_2$ and $E_3$ shall be of steel. Joists for classes C, $D_1$ and $D_2$ may be either of wood or steel as specified. Steel joists shall be securely fastened to the cross floorbeams, and steel stringers shall preferably be riveted to the webs of floorbeams by means of connection angles at least $\frac{5}{16}$ in. thick.

17. **End Spacers for Stringers.**—Where end floorbeams cannot be used, stringers resting on masonry shall have cross-frames at their ends. These frames shall be riveted to girder or truss shoe where practicable.

18. **Wooden Joists.**—Wooden floor joists shall be spaced not more than $2\frac{1}{2}$ ft. centers, and shall lap by each other so as to have a full bearing on the floorbeams, and shall be separated $\frac{1}{2}$ in. for free circulation of air. Their width shall not be less than 3 in., or one-fourth the depth in width. The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists in feet divided by four feet. No impact shall be considered in the design of wooden joists, planks or ties. Oak, longleaf yellow pine and Oregon fir shall be designed for a safe bending of 1,500 lb. per sq. in., bearing across the fiber of 400 lb. per sq. in., and shearing along the grain of 140 lb. per sq. in. Outside joists shall be designed for the same live loads as the intermediate joists.

19. **Steel Joists.**—Steel I-beams when used as joists shall have a depth of not less than one-thirtieth of the span, and one-twentieth of the span when used as track stringers. The proportion of the concentrated live load carried by one joist shall be taken equal to the spacing of the joists in feet divided by four feet when timber flooring is used, and divided by six feet when a reinforced concrete or other rigid floor is used. Outside joists shall be designed for the same live loads as the intermediate joists.

20. **Floor Plank.**—For single thickness the roadway planks shall not be less than 3 in. thick nor less than one-eighth of the distance between centers of joists, and shall be laid transversely with $\frac{1}{2}$ in. openings and securely spiked to each joist. All plank shall be laid with heart side down. When an additional wearing surface is required it shall be $1\frac{1}{2}$ in. thick, and the lower planks of a minimum thickness of 3 in. shall be laid diagonally with $\frac{1}{2}$ in. openings.

21. Footwalk plank shall be not less than 2 in. thick nor more than 6 in. wide, spaced with $\frac{3}{8}$ in. openings.

All plank shall be laid with heart side down, shall have full and even bearing on and be firmly attached to the joists.

22. **Wheel Guards.**—Wheel guards of a cross-section of not less than 6 in. by 4 in. shall be provided on each side of the roadway. They shall be spliced with half-and-half joints with 6 in. lap, and shall be bolted to the stringers or joist with $\frac{5}{8}$ in. bolts, spaced not to exceed 5 ft. apart.

23. **Solid Floor.**—For bridges of classes A and B a solid floor, consisting of wooden blocks, brick, stone, asphalt, etc., on a concrete bed is recommended. For this case the floor shall consist of buckle plates or corrugated sections or reinforced concrete slabs, and a waterproof concrete (bitumen or cement) bed not less than 3 in. thick for the roadway and 2 in. thick for the footwalk, over the highest point to be covered, not counting rivet or bolt heads. The floor shall be laid with a slope of at least one inch in 10 ft.

**Reinforced Concrete Floor.**—See specifications for reinforced concrete floor in "General Specifications for Concrete Highway Bridges and Foundations," Appendix II, and for the distribution of loads on slabs see Chapter IX.

24. Buckle plates shall not be less than $\frac{5}{16}$ in. thick for the roadway and $\frac{1}{4}$ in. thick for the footwalk. The crown of the plates shall not be less than 2 in.

25. For solid floor the curb holding the paving and acting as a wheel guard on each side of the roadway shall be of stone or steel projecting about 6 in. above the finished paving at the gutter. The curb shall be so arranged that it can be removed and replaced when worn or injured. There shall also be a metal edging strip on each side of the footwalk to protect and hold the paving in place.

26. **Drainage.**—Provision shall be made for drainage clear of all parts of the metal work.

27. **Floor of Classes $E_1$, $E_2$, and $E_3$.**—The floors of classes $E_1$, $E_2$, and $E_3$ shall consist of cross-ties not less than 6 in. by 6 in. for stringers spaced $6\frac{1}{2}$ ft.; and larger for greater spacings, they shall be spaced with openings not exceeding 6 in., shall be notched down $\frac{1}{2}$ in., and secured to the supporting stringers by $\frac{3}{4}$ in. bolts spaced not over 6 ft. apart. The ties shall extend the full width of the bridge on deck bridges, and every other tie shall extend the full width in through bridges to carry the footwalk. Ties shall be designed for the same allowable unit stresses as wooden joists.

There shall be guard timbers not less than 6 in. by 6 in., or 5 in. by 7 in., on each side of each track, with their inner faces not less than 9 in. from the center of the rail. They shall be notched 1 in. over every tie, and shall be spliced over a tie with a half-and-half joint with 6 in. lap. Each guard timber shall be fastened to every third tie and at each splice with a $\frac{3}{4}$ in. bolt. All heads or nuts on the upper faces of ties or guards shall be countersunk below the surface of the wood.

## PART II. LOADS.

28. **Dead Load.**—The dead load will consist of (1) the weight of the metal, and (2) the weight of the timber in the floor, or of the material other than steel. In determining the dead load the weight of oak or other hard wood shall be taken at $4\frac{1}{2}$ lb. per foot board measure, and the weight of pine or other soft woods at $3\frac{1}{2}$ lb. per foot; the weight of asphalt at 130 lb., of concrete and paving brick at 150 lb., and of granite at 160 lb .per cu. ft.

The rails, fastenings, splices and guard timbers of street railway tracks shall be assumed to weigh not less than 100 lb. per lineal foot of track.

29. **Live Load.**—The bridges of different classes shall be designed to carry, in addition to their own weight and that of the floor, a moving load, either uniform or concentrated, or both, as specified below, placed so as to give the greatest stress in each member.

*Class A. For City Traffic.*—For the floor and its supports, on any part of the roadway or on each of the street car tracks, a concentrated load of 24 tons on two axles 10 ft. centers and 5 ft. gage (assumed to occupy 12 ft. in width for a single line or 22 ft. for a double line), and upon the remaining portion of the floor, a load of 125 lb. per sq. ft. and a concentrated load as for class $D_1$. Sidewalks a load of 100 lb. per sq. ft.

Loads for the trusses as per Table I.

*Class B. For Suburban or Interurban Traffic.*—For the floor and its supports, on any part of the roadway, a concentrated load of 12 tons on two axles 10-ft. centers and 5-ft. gage (assumed to occupy a width of 12 ft.), or on each street car track a concentrated load of 24 tons on two axles 10-ft. centers; and on the remaining portion of the floor, a load of 125 lb. per sq. ft. and a concentrated load as for class $D_1$. Sidewalks a load of 100 lb. per sq. ft.

Loads for the trusses as per Table I.

*Class C. For Highway and Light Interurban Traffic.*—For the floor and its supports, on any part of the roadway, a concentrated load of 12 tons on two axles 10-ft. centers and 5-ft. gage (assumed to occupy a width of 12 ft.), or on each street car track a concentrated load of 18 tons on two axles 10-ft. centers; and upon the remaining portion of the floor, a load of 125 lb. per sq. ft. and a concentrated load as for class $D_1$. Sidewalks a load of 100 lb. per sq. ft.

Loads for the trusses as per Table I.

*Class $D_1$. Heavy Country Bridges.*—For the floor and its supports, a load of 125 lb. per sq. ft. of total floor surface or a 20-ton motor truck with axles spaced 12 ft. and wheels with 6 ft. centers, with 14 tons on rear axle and 6 tons on front axle. The truck to occupy a space 10 ft. wide and 32 ft. long. The rear wheels to have a width of 20 in.

Loads for the trusses as per Table I. No bridge, however, to be designed for a load of less than 1,000 lb. per lineal foot of bridge.

*Class $D_2$. Oridnary Country Bridges.*—For the floor and its supports, a load of 100 lb. per sq. ft. of total floor surface of a 15-ton motor truck with axles spaced 10 ft. and wheels with 6 ft. centers, and occupying a space 10 ft. wide and 30 ft. long, with 10 tons on rear axle and 5 tons on front axle, and with rear wheels 15 in. wide.

Loads for the trusses as per Table I. No bridge, however, to be designed for a load of less than 800 lb. per lineal foot of bridge.

*Class $E_1$. For Heavy Electric Railways Only.*—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are loaded with a load of 40,000 lb., making a total of 160,000 lb. Or a uniform load of 6,000 lb. per lineal foot for all spans up to 50 ft., reduced to 4,500 lb. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

*Class $E_2$. For Medium Electric Railways Only.*—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5-ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15-ft. centers. The axles are loaded with a load of 25,000 lb., making a total load of 100,000 lb. Or a uniform load of 3,500 lb. per lineal foot for all spans up to 50 ft., reduced to 2,000 lb. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

*Class $E_3$. For Light Electric Railways Only.*—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5-ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft.-centers. The axles are loaded with a load of 20,000 lb. making a total load of 80,000 lb. Or a uniform load of 2,500 lb. per lineal foot for all spans up to 50 ft., reduced to 1,500 lb. per lineal foot for spans of 200 ft. and over, and proportionately for intermediate spans.

TABLE I.

LIVE LOADS FOR THE TRUSSES

| Span in Feet. | Class A. | | Class B. | | Class C. | | Class $D_1$. | Class $D_2$. |
|---|---|---|---|---|---|---|---|---|
| | Pounds per Lineal Foot of Each Car Track. | Pounds per Square Foot of Remaining Floor Surface. | Pounds per Lineal Foot of Each Car Track. | Pounds per Square Foot of Remaining Floor Surface. | Pounds per Lineal Foot of Each Car Track. | Pounds per Square Foot of Remaining Floor Surface. | Pounds per Square Foot of Floor Surface. | Pounds per Square Foot of Floor Surface. |
| Up to 30........ | 1,800 | 125 | 1,800 | 125 | 1,800 | 125 | 125 | 100 |
| 80........ | 1,800 | 105 | 1,800 | 85 | 1,200 | 85 | 85 | 75 |
| 160........ | 1,440 | 88 | 1,440 | 68 | 1,080 | 68 | 68 | 60 |
| 200 and over | 1,200 | 80 | 1,200 | 60 | 1,000 | 60 | 60 | 50 |
| Loads for intermediate spans to be proportional. | | | | | | | | |

30. **Wind Loads.**—The lateral bracing in the unloaded chords of truss bridges shall be designed for a lateral wind load of 150 lb. per lineal foot of bridge, considered as a moving load. The lateral bracing in the loaded chords of truss bridges shall be designed for a lateral wind load of 300 lb. per lineal foot of bridge, considered as a moving load. For spans over 300 ft. each of the above loadings shall be increased 10 lb. for each 20 ft. increase in span. In highway bridges not carrying electric cars the end-posts of through and deck bridges and the intermediate posts of through bridges shall be designed for a combination (1) of the dead load stresses and the total live load stresses; or (2) of the dead load stresses, the live load stresses, the impact and centrifugal stresses, and one-half the total wind load stresses. In low truss bridges and plate girders not carrying electric cars the wind load on the unloaded chord may be omitted and the lateral bracing be designed for a lateral wind load of 300 lb. per lineal foot treated as a moving load. In bridges with sway bracing one-half of the wind load may be assumed to pass to the lower chord through the sway bracing.

End-posts of riveted through trusses with end floorbeams riveted rigidly at ends, shall be assumed as fixed at lower end.

31. In trestle towers the bracing and columns shall be designed to resist the following lateral forces, in addition to the stresses due to dead and live loads: The trusses loaded or unloaded, the lateral pressures specified above; and a lateral pressure of 100 lb. for each vertical lineal foot of trestle bent.

32. **Temperature.**—Stresses due to a variation in temperature of 150 degrees shall be provided for (§ 81).

33. **Centrifugal Force of Train.**—Structures located on curves shall be designed for the centrifugal force of the live load acting at the top of the rail. The centrifugal force shall be calculated by the following formula: $C = 0.03\, W \cdot D$; where $C$ = centrifugal force in lb.; $W$ = weight of train in lb.; and $D$ = degree of curvature.

34. **Longitudinal Forces.**—The stresses produced in the bracing of the trestle towers, in any members of the trusses, or in the attachments of the girders or trusses to their bearings, by sud-

denly stopping the maximum electric car trains on any part of the work must be provided for; the coefficient of friction of the wheels on the rails being assumed as 0.20.

35. All parts shall be so designed that the stresses coming upon them can be accurately calculated.

## PART III. UNIT STRESSES AND PROPORTION OF PARTS.

36. **Unit Stresses.**—All parts of the structure shall be proportioned so that the sum of the maximum stresses shall not exceed the following amounts in lb. per sq. in., except as modified by § 45 and § 48.

**Impact.**—The dynamic increment of the live load stress shall be added to the maximum live load stresses as follows:

For the floor and its supports including floor slabs, floor joist, floorbeams and hangers, 30 per cent.

For all truss members other than the floor and its supports, the impact increment shall be $I = 100/(L + 300)$, where $L$ = length of span for simple highway spans (for trestle bents, towers, movable bridges, arch and cantilever bridges, and for bridges carrying electric trains, $L$ shall be taken as the loaded length of the bridge in feet producing maximum stress in the member).

Impact shall not be added to the stresses produced by longitudinal, centrifugal and lateral or wind forces.

37. **Tension.**—Axial tension on net section .......... 16,000

The lengths of riveted tension members in horizontal or inclined positions shall not exceed 200 times their radius of gyration about the horizontal axis. The horizontal projection of the unsupported portion of the member is to be considered as the effective length.

38. **Compression.**—Axial compression on gross section .......... $16{,}000 - 70 \cdot l/r$ with a maximum of 14,000 lb.; where "$l$" is the length of member in inches and "$r$" is the least radius of gyration in inches.

No compression member, however, shall have a length exceeding 100 times its least radius of gyration for main members or 120 times for laterals for classes A, B, C, $E_1$, $E_2$, and $E_3$; or 125 times its least radius of gyration for main members or 150 times for laterals for classes $D_1$ and $D_2$.

39. **Bending.**—Bending: on extreme fibers of rolled shapes, built sections and girders;
net section .......... 16,000
on cast iron .......... 3,000
on extreme fibers of pins .......... 24,000

40. **Shearing.**—Shearing: shop driven rivets and pins .......... 12,000
field driven rivets and turned bolts .......... 10,000
plate girder webs; gross section .......... 10,000
cast iron .......... 1,500

41. **Bearing.**—Bearing; shop driven rivets and pins .......... 24,000
field driven rivets and turned bolts .......... 20,000
cast iron .......... 12,000
granite masonry and Portland cement concrete .......... 600
sandstone and limestone .......... 400
expansion rollers; per lineal inch .......... $600d$
cast iron expansion rockers; per lineal inch .......... $300d$
where "$d$" is the diameter of the roller in inches.

Rivets shall not be used in direct tension, except for lateral bracing where unavoidable; in which case the value for direct tension on the rivet shall be taken the same as for single shear.

42. **Alternate Stresses.**—Members subject to alternate stresses of tension and compression shall be proportioned for the stresses giving the largest section. If the alternate stresses occur in succession during the passage of one train, as in stiff counters, each stress shall be increased by 50 per cent of the smaller. The connections shall in all cases be proportioned for the sum of the stresses.

43. **Angles in Tension.**—When single-angle members subject to direct tension are fastened by one leg, only seventy-five per cent of the net area shall be considered effective. Angles with lug angle connections shall not be considered as fastened by both legs.

44. **Net Section.**—In members subject to tensile stresses full allowance shall be made for reduction of section by rivet-holes, screw-threads, etc. In calculating net area the rivet-holes shall be taken as having a diameter ⅛ in. greater than the normal size of rivet.

The net section of riveted members shall be the least area which can be obtained by deducting from the gross sectional area the areas of holes cut by any plane perpendicular to the axis of the member and parts of the areas of other holes on one side of the plane, within a distance of 4 inches, and which are on other gage lines than those of the holes cut by the plane, the parts being determined by the formula:

$$A(1 - p/4),$$

in which $A$ = the area of the hole, and

$p$ = the distance in inches of the center of the hole from the plane.

45. **Long Span Bridges.**—For long span bridges, where the ratio of the length to width of span is such that it makes the top chords acting as a whole, a longer column than the segments of the chords, the chord shall be proportioned for the greater length.

46. **Wind Stresses.**—The stresses in truss members or trestle posts from assumed wind forces need not be considered except as follows:

1. When the direct wind stresses per square inch in any member exceed 25 per cent of the stresses due to dead and live loads in the same member. The section shall then be increased until the total unit stress shall not exceed by more than 25 per cent the maximum allowable stress for dead and live loads.

2. When the wind stress alone or in combination with a possible temperature stress can neutralize or reverse the stresses in the member.

When both direct and flexural stresses due to wind are considered 50 per cent may be added to allowable stresses for dead and live loads, provided the area thus obtained is not less than required for dead and live loads alone, or for dead, live and direct wind loads designed as in § 46.

47. **Combined Stresses.**—Members subjected to direct and bending stresses shall be designed so that the greatest fiber stress shall not exceed the allowable unit stress on the member.

48. **Stress Due to Weight and Eccentric Loading.**—If the fiber stress due to weight and eccentric loading on any member exceeds 10 per cent of the allowable unit stress on the member, such excess must be considered in proportioning the member. See § 46.

49. **Counters.**—Counters in bridges carrying electric cars shall be designed so that an increase of the live load of 25 per cent will not increase the stress in the counters more than 25 per cent.

50. **Design of Plate Girders.**—Plate girders shall be proportioned either by the moment of inertia of their net section; or by assuming that the flanges are concentrated at their centers of gravity, in which case one-eighth of the gross section of the web, if properly spliced, may be used as flange section. The thickness of web plates shall be not less than $\frac{5}{16}$ in., nor less than 1/160 of the unsupported distance between flange angles.

**Compression Flanges.**—In beams and plate girders the compression flanges shall have the same gross section as the tension flanges. Through plate girders shall have their top flanges stayed at each end of every floorbeam, or in case of solid floors, at distances not exceeding 12 ft., by knee braces or gusset plates. The stress per sq. in. in compression flange of any beam or girder shall not exceed $16,000 - 150\, l/b$, where $l$ = unsupported distance and $b$ = width of flange.

51. **Web Stiffeners.**—There shall be web stiffeners, generally in pairs, over bearings, at points of concentrated loading, and at other points where the thickness of the web is less than $\frac{1}{60}$ of the unsupported distance between flange angles. The distance between stiffeners shall not exceed that given by the following formula, with a maximum limit of six feet (and not greater than the clear depth of the web): $d = t\,(12,000 - s)/40$.

Where $d$ = clear distance, between stiffeners of flange angles; $t$ = thickness of web; $s$ = shear in lb. per sq. in.

The stiffeners at ends and at points of concentrated loads shall be proportioned by the formula of paragraph 38, the effective length being assumed as one-half the depth of girders. End stiffeners and those under concentrated loads shall be on fillers and have their outstanding legs as wide as the flange angles will allow and shall fit tightly against them. Intermediate stiffeners may be offset or on fillers, and their outstanding legs shall be not less than one-thirtieth of the depth of girder, plus 2 in.

52. **Flange Rivets.**—The flanges of plate girders shall be connected to the web with a sufficient number of rivets to transfer the total shear at any point in a distance equal to the effective depth of the girder at that point combined with any load that is applied directly on the flange. The wheel loads, where the ties rest on the flanges, shall be assumed to be distributed over three ties.

53. **Depth Ratios.**—Trusses shall preferably have a depth of not less than one-tenth of the span. Plate girders and rolled beams, used as girders, shall preferably have a depth of not less than one-twelfth of the span. If shallower trusses, girders or beams are used, the section shall be increased so that the maximum deflection will not be greater than if the above limiting ratios had not been exceeded. For steel joists and track stringers, see § 19.

54. **Low Trusses.**—Riveted low trusses shall have top chords composed of a double web member with cover plate. The top chords shall be stayed against lateral bending by means of brackets or knee braces rigidly connected to the floorbeam at intervals not greater than twelve times the width of the cover plate. The posts shall be solid web members. The floorbeams shall be riveted, preferably above the lower chord. Pin-connected low truss bridges shall not be used.

55. **Rolled Beams.**—Rolled beams shall be designed by using their moments of inertia. The webs of rolled beams and plate girders shall be assumed to take all the shear.

## PART IV. DETAILS OF DESIGN

### General Requirements.

56. **Open Sections.**—Structures shall be so designed that all parts will be accessible for inspection, cleaning and painting.

57. **Water Pockets.**—Pockets or depressions which would hold water shall have drain holes, or be filled with waterproof material.

58. **Symmetrical Sections.**—Main members shall be so designed that the neutral axis will be as nearly as practicable in the center of section, and the neutral axes of intersecting main members of trusses shall meet at a common point.

59. **Counters.**—Rigid counters are preferred; and where subject to reversal of stress shall have riveted connections to the chords. Adjustable counters shall have open turn-buckles.

60. **Strength of Connections.**—The strength of connections shall be sufficient to develop the full strength of the member, even though the computed stress is less, the kind of stress to which the member is subjected being considered.

61. **Minimum Thickness.**—The minimum thickness of metal shall be $\frac{5}{16}$ in. in classes A, B, C, $E_1$, $E_2$ and $E_3$, except for fillers; and $\frac{1}{4}$ in. in classes $D_1$ and $D_2$, except for fillers and webs of channels. Webs of channels for classes $D_1$ and $D_2$ may have a minimum thickness of 0.20 in. The minimum angle shall be 2 in. $\times$ 2 in. $\times$ $\frac{1}{4}$ in. The minimum rod shall have an area of at least 1 sq. in., in all classes except $D_1$ and $D_2$, which shall have no rods less than $\frac{3}{4}$ in. in diameter. Webs of plate girders shall not be less than $\frac{5}{16}$ in.

62. **Pitch of Rivets.**—The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3 in. for $\frac{7}{8}$-in. rivets, $2\frac{1}{2}$ in. for $\frac{3}{4}$-in rivets, and 2 in. for $\frac{5}{8}$-in. rivets. The maximum pitch in the line of stress for members composed of plates and shapes shall be 16 times the thickness of the thinnest outside plate or 6 in. For angles with two gage lines and rivets staggered, the maximum shall be twice the above in each line. Where two or more plates are used in contact, rivets not more than 12 in. apart in either direction shall be used to hold the plates well together. In tension members composed of two angles in contact, a pitch of 12 in. will be allowed for riveting the angles together.

63. **Edge Distance.**—The minimum distance from the center of any rivet hole to a sheared edge shall be $1\frac{1}{2}$ in. for $\frac{7}{8}$-in. rivets, $1\frac{1}{4}$ in. for $\frac{3}{4}$-in. rivets, and $1\frac{1}{8}$ in. for $\frac{5}{8}$-in. rivets, and to a rolled edge $1\frac{1}{4}$, $1\frac{1}{8}$ and 1 in., respectively. The maximum distance from any edge shall be eight times the thickness of the plate, but shall not exceed 6 in.

64. **Maximum Diameter.**—The diameter of the rivets in any angle carrying calculated stress shall not exceed one-quarter the width of the leg in which they are driven. In minor parts $\frac{7}{8}$-in. rivets may be used in 3-in. angles, $\frac{3}{4}$-in. rivets in $2\frac{1}{2}$-in. angles, and $\frac{5}{8}$-in. rivets in 2-in. angles.

65. **Long Rivets.**—Rivets carrying calculated stress and whose grip exceeds four diameters shall be increased in number at least one per cent for each additional $\frac{1}{16}$-in. of grip.

66. **Pitch at Ends.**—The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets, for a length equal to one and one-half times the maximum width of member.

67. **Compression Members.**—In compression members the metal shall be concentrated as much as possible in webs and flanges. The thickness of each web shall be not less than one-thirtieth of the distance between its connections to the flanges. Cover plates shall have a thickness not less than one-fortieth of the distance between rivet lines.

68. **Minimum Angles.**—Flanges of girders and built members without cover plates shall have a minimum thickness of one-twelfth of the width of the outstanding leg.

69. **Batten Plates.**—The open sides of all compression members shall be stayed by batten plates at the ends and diagonal lattice-work at intermediate points. The batten plates must be placed as near the ends as practicable, and shall have a length not less than the greatest width of the member or $1\frac{1}{2}$ times its least width.

70. **Lacing Bars.**—The lacing of compression members shall be proportioned to resist a shearing stress of $2\frac{1}{2}$ per cent of the direct stress. The minimum width of lacing bars shall be $1\frac{3}{4}$ in. for members 6 in. in width, 2 in. for members 9 in. in width, $2\frac{1}{4}$ in. for members 12 in. in width, $2\frac{1}{2}$ in. for members 15 in. in width, nor 3 in. for members 18 in. and over in width. Single lacing bars shall have a thickness not less than one-fortieth, or double lacing bars connected by a rivet at the intersection, not less than one-sixtieth of the distance between the rivets connecting them

to the members. They shall be inclined at an angle not less than 60° to the axis of the member for single lacing, nor less than 45° for double lacing with riveted intersections.

71. **Spacing of Lacing Bars.**—Lacing bars shall be so spaced that the portion of the flange included between their connection shall be as strong as the member as a whole. The pitch of the lacing bars must not exceed the width of the channel plus nine inches.

72. **Rivets in Flanges.**—Five-eighths-inch rivets shall be used for lacing flanges less than 2½ in. wide; ¾-in. for flanges from 2½ to 3½ in. wide; ⅞-in. rivets shall be used in flanges 3½ in. and over. Lacing bars with two rivets shall be used for flanges over 5 in. wide.

73. **Splices.**—In compression members joints with abutting faces planed shall be placed as near the panel points as possible, and must be spliced on all sides with at least two rows of rivets on each side of the joint. Joints with abutting faces not planed shall be fully spliced. Joints in tension members shall be fully spliced.

74. **Pin Plates.**—Where necessary, pin-holes shall be reinforced by plates, some of which must be of the full width of the member, so the allowed pressure on the pins shall not be exceeded, and so the stresses shall be properly distributed over the full cross-section of the members. These reinforcing plates must contain enough rivets to transfer their proportion of the bearing pressure, and at least one plate on each side shall extend not less than 6 in. beyond the edge of the nearest batten plate.

75. **Riveted Tension Members.**—Riveted tension members shall have an effective section through the pin-holes 25 per cent in excess of the net section of the member, and back of the pin at least 75 per cent of the net section through the pin-hole.

76. **Pins.**—Pins shall be long enough to insure a full bearing of all the parts connected upon the turned body of the pin. The diameter of the pin shall not be less than ¾ of the depth of any eye-bar attached to it. They shall be secured by chambered Lomas nuts or be provided with washers if solid nuts are used. The screw ends shall be long enough to admit of burring the threads.

77. **Filling Rings.**—Members packed on pins shall be held against lateral movement.

78. **Bolts.**—Where members are connected by bolts, the turned body of these bolts shall be long enough to extend through the metal. A washer at least ½ in. thick shall be used under the nut. Bolts shall not be used in place of rivets except by special permission. Heads and nuts shall be hexagonal.

79. **Indirect Splices.**—Where splice plates are not in direct contact with the parts which they connect, rivets shall be used on each side of the joint in excess of the number theoretically required to the extent of one-third of the number for each intervening plate.

80. **Fillers.**—Rivets carrying stress and passing through fillers shall be increased 50 per cent in number; and the excess rivets, when possible, shall be outside of the connected member.

81. **Expansion.**—Provision for expansion to the extent of ⅛ in. for each 10 ft. shall be made for all bridge structures. Efficient means shall be provided to prevent excessive motion at any one point (§ 32).

82. **Expansion Bearings.**—Spans of 60 ft. and over resting on masonry shall have turned rollers or rockers at one end; and those of less length shall be arranged to slide on smooth metal surfaces.

83. **Fixed Bearings.**—Movable bearings shall be designed to permit motion in one direction only. Fixed bearings shall be firmly anchored to the masonry (§ 87).

84. **Rollers.**—Expansion rollers shall be not less than 3 in. in diameter for spans of 100 feet or less, and shall be increased 1 in. for each 100 ft. additional. They shall be coupled together with substantial side bars, which shall be so arranged that the rollers can be readily cleaned.

85. **Bolsters.**—Bolsters or shoes shall be so constructed that the load will be distributed over the entire bearing.

86. **Pedestals and Bed Plates.**—Built pedestals shall be made of plates and angles. All bearing surfaces of the base plates and vertical webs must be planed. The vertical webs must be secured to the base by angles having two rows of rivets in the vertical legs. No base plate or web connecting angle shall be less in thickness than ⅜ in. The vertical webs shall be of sufficient height and must contain material and rivets enough to practically distribute the loads over the bearings or rollers.

Where the size of the pedestal permits, the vertical webs must be rigidly connected transversely.

The details of cast iron or cast steel shoes shall be subject to the special approval of the engineer. The vertical webs of cast iron rockers and pedestals shall be designed for an allowable unit stress of $9{,}000 - 40l/r$, where $h$ = height and $r$ = radius of gyration of vertical web, both in inches.

87. All the bed-plates and bearings under fixed and movable ends must be fox-bolted to the masonry; for trusses, these bolts must not be less than $1\frac{1}{4}$ in. diameter; for plate and other girders, not less than $\frac{7}{8}$ in. diameter.

88. **Wall Plates.**—Wall plates may be cast or built up; and shall be so designed as to distribute the load uniformly over the entire bearing. They shall be secured against displacement.

89. **Anchorage.**—Anchor bolts for viaduct towers and similar structures shall be long enough to engage a mass of masonry the weight of which is at least one and one-half times the uplift (§ 11).

90. **Inclined Bearings.**—Bridges on an inclined grade without pin shoes shall have the sole plates beveled so that the masonry and expansion surfaces may be level.

91. **Camber.**—Truss spans shall be given a camber by making the panel length of the top chords, or their horizontal projections, longer than the corresponding panels of the bottom chord in the proportion of $\frac{1}{8}$ in. in 10 ft. Plate girder spans need not be cambered.

92. **Eye-bars.**—The eye-bars composing a member shall be so arranged that adjacent bars shall not have their surfaces in contact; they shall be as nearly parallel to the axis of the truss as possible, the maximum inclination of any bar being limited to one inch in 16 ft.

## PART V. MATERIALS AND WORKMANSHIP.

### Material.

93. **Process of Manufacture.**—Steel shall be made by the open-hearth process and shall comply with the standard specifications for structural steel for bridges adopted by the American Society for Testing Materials.

(Sections 94 to 117 inclusive cover the American Society for Testing Materials Specifications for Steel for Bridges, see Ketchum's Structural Engineer's Handbook).

118. **Timber.**—The timber shall be strictly first-class spruce, white pine, Douglas fir, Southern yellow pine, or white oak bridge timber; sawed true and out of wind, full size, free from wind shakes, large or loose knots, decayed or sapwood, wormholes or other defects impairing its strength or durability.

### Workmanship.

119. **General.**—All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works.

120. **Straightening Material.**—Material shall be thoroughly straightened in the shop, by methods that will not injure it, before being laid off or worked in any way.

121. **Finish.**—Shearing shall be neatly and accurately done and all portions of the work exposed to view neatly finished.

122. **Size of Rivets.**—The size of rivets, called for on the plans, shall be understood to mean the actual size of the cold rivet before heating.

123. **Rivet Holes.**—When general reaming is not required the diameter of the punch shall not be more than $\frac{1}{16}$ in. greater than the diameter of the rivet; nor the diameter of the die more than $\frac{1}{8}$ in. greater than the diameter of the punch. Material more than $\frac{3}{4}$ in. thick shall be sub-punched and reamed or drilled from the solid.

124. **Punching.**—All punching shall be accurately done. Drifting to enlarge unfair holes will not be allowed. If the holes must be enlarged to admit the rivet, they shall be reamed. Poor matching of holes will be cause for rejection.

125. **Sub-punching and Reaming.**—Where reaming is required, the punch used shall have a diameter not less than $\frac{3}{16}$ in. smaller than the nominal diameter of the rivet. Holes shall then be reamed to a diameter not more than $\frac{1}{16}$ in. larger than the nominal diameter of the rivet. All reaming shall be done with twist drills. (§ 140).

126. **Reaming After Assembling.**—When general reaming is required it shall be done after the pieces forming one built member are assembled and firmly bolted together. If necessary to take the pieces apart for shipping and handling, the respective pieces reamed together shall be so marked that they may be reassembled in the same position in the final setting up. No interchange of reamed parts will be allowed.

127. **Edge Planing.**—Sheared edges or ends shall, when required, be planed at least $\frac{1}{8}$ in.

128. **Burrs.**—The outside burrs on reamed holes shall be removed.

129. **Assembling.**—Riveted members shall have all parts well pinned up and firmly drawn together with bolts, before riveting is commenced. Contact surfaces to be painted.

130. **Lacing Bars.**—Lacing bars shall have neatly rounded ends, unless otherwise called for.

131. **Web Stiffeners.**—Stiffeners shall fit neatly between flanges of girders. Where tight fits are called for, the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

132. **Splice Plates and Fillers.**—Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$ in. of flange angles.

133. **Web Plates.**—Web plates of girders, which have no cover plates, shall be flush with the backs of angles or project above the same not more than $\frac{1}{8}$ in., unless otherwise called for. When web plates are spliced, not more than $\frac{1}{4}$ in. clearance between ends of plates will be allowed.

134. **Connection Angles.**—Connection angles for floorbeams and stringers shall be flush with each other and correct as to position and length of girder. In case milling (of all such angles) is needed or is required after riveting, the removal of more than $\frac{1}{16}$ in. from their thickness will be cause for rejection.

135. **Rivets.**—Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

136. **Riveting.**—Rivets shall look neat and finished, with heads of approved shape, full and of equal size. They shall be central on shank and grip the assembled pieces firmly. Recupping and calking will not be allowed. Loose, burned or otherwise defective rivets shall be cut out and replaced. In cutting out rivets, great care shall be taken not to injure the adjacent metal. If necessary, they shall be drilled out.

137. **Turned Bolts.**—Wherever bolts are used in place of rivets which transmit shear, the holes shall be reamed parallel and the bolts turned to a driving fit. A washer not less than $\frac{1}{4}$ in. thick shall be used under nut.

138. **Members to be Straight.**—The several pieces forming one built member shall be straight and fit closely together, and finished members shall be free from twists, bends or open joints.

139. **Finish of Joints.**—Abutting joints shall be cut or dressed true and straight and fitted close together, especially where open to view. In compression joints, depending on contact bearing, the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned.

140. **Field Connections.**—Holes for floorbeam and stringer connections shall be sub-punched and reamed according to paragraph 125, to a steel templet one inch thick. (If required, all other field connections, except those for laterals and sway bracing, shall be assembled in the shop and the unfair holes reamed; and when so reamed, the pieces shall be match-marked before being taken apart.)

141. **Eye-bars.**—Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body when tested to rupture. The thickness of head and neck shall not vary more than $\frac{1}{16}$ in. from that specified.

142. **Boring Eye-bars.**—Before boring, each eye-bar shall be properly annealed and carefully straightened. Pin-holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that, when placed together, pins $\frac{1}{32}$ in. smaller in diameter than the pin-holes can be passed through the holes at both ends of the bars at the same time without forcing.

143. **Pin-Holes.**—Pin-holes shall be bored true to gages, smooth and straight; at right angles to the axis of the member and parallel to each other, unless otherwise called for. The boring shall be done after the member is riveted up.

144. **Variation in Pin-Holes.**—The distance center to center of pin-holes shall be correct within $\frac{1}{32}$ in., and the diameter of the holes not more than $\frac{1}{50}$ in. larger than that of the pin, for pins up to 5-in. diameter, and $\frac{1}{32}$ in. for larger pins.

145. **Pins and Rollers.**—Pins and rollers shall be accurately turned to gages and shall be straight and smooth and entirely free from flaws.

146. **Screw Threads.**—Screw threads shall make tight fits in the nuts and shall be U. S. standard, except above the diameter of $1\frac{3}{8}$ in., when they shall be made with six threads per inch.

147. **Annealing.**—Steel, except in minor details, which has been partially heated, shall be properly annealed.

148. **Steel Castings.**—All steel castings shall be annealed.

149. **Welds.**—Welds in steel will not be allowed except to remedy minor defects in steel castings.

150. **Bed Plates.**—Expansion bed plates shall be planed true and smooth. Cast wall plates shall be planed top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

151. **Pilot Nuts.**—Pilot and driving nuts shall be furnished for each size of pin, in such numbers as may be ordered.

152. **Field Rivets.**—Field rivets shall be furnished to the amount of 15 per cent plus ten rivets in excess of the nominal number required for each size.

153. **Shipping Details.**—Pins, nuts, bolts, rivets and other small details shall be boxed or crated.

154. **Weight.**—The weight of every piece and box shall be marked on it in plain figures.

155. **Weight Paid For.**—The payment for pound price contracts shall be based on scale weights of the metal in the fabricated structure, including field rivets 15 per cent plus 10 rivets in excess of the number nominally required. The weight of the shop coat of paint, field paint, cement, fitting up bolts, pilot nuts, driving caps, boxes and barrels used for packing, and material used in supporting members on cars shall be excluded. If the scale weight is more than $2\frac{1}{2}$ per cent under the computed weight it may be cause for rejection. The greatest allowable variation of the total scale weight of any structure from the weights computed from the approved shop drawings shall be $1\frac{1}{2}$ per cent. Any weight in excess of $1\frac{1}{2}$ per cent above the computed weight shall not be paid for. The weights of rolled shapes and plates up to and including 36 in. in width shall be computed on the basis of their normal weights and dimensions, as shown on the approved drawings, deducting for all copes, cuts and open holes. With plates more than 36 in. in width, the weights are to be calculated in the same manner as for plates 36 in. and under, except that one-half the percentage of overrun given in the Standard Specifications for Structural Steel for Bridges of the American Society for Testing Materials shall be added. The weight of heads of shop driven rivets shall be included in the computed weight. The weights of castings shall be computed from the dimensions shown on the approved drawings, with an addition of 10 per cent for fillets and overrun.

## SHOP PAINTING.

156. **Cleaning.**—Steel work, before leaving the shop, shall be thoroughly cleaned and given one good coating of pure linseed oil, or such paint as may be called for, well worked into all joints and open spaces.

157. **Contact Surfaces.**—In riveted work, the surfaces coming in contact shall each be painted before being riveted together.

158. **Inaccessible Surfaces.**—Pieces and parts which are not accessible for painting after erection, including tops of stringers, eye-bar heads, ends of posts and chords, etc., shall have a good coat of paint before leaving the shop.

159. **Condition of Surfaces.**—Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

160. **Machine-finished Surfaces.**—Machine-finished surfaces shall be coated with white lead and tallow before shipment or before being put out into the open air.

## INSPECTION AND TESTING AT THE SHOP AND MILL.

161. **Facilities for Shop Inspection.**—The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of workmanship at the shop where material is manufactured. He shall furnish a suitable testing machine for testing full-sized members, if required.

162. **Starting Work in Shop.**—The purchaser shall be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect material and workmanship.

163. **Copies of Mill Orders.**—The purchaser shall be furnished complete copies of mill orders, and no material shall be rolled, nor work done, before the purchaser has been notified where the orders have been placed, so that he may arrange for the inspection.

164. **Facilities for Mill Inspection.**—The manufacturer shall furnish all facilities for inspecting and testing the weight and quality of all material at the mill where it is manufactured. He shall furnish a suitable testing machine for testing the specimens, as well as prepare the pieces for the machine, free of cost.

165. **Access to Mills.**—When an inspector is furnished by the purchaser to inspect material at the mills, he shall have full access, at all times, to all parts of mills where material to be inspected by him is being manufactured.

166. **Access to Shop.**—When an inspector is furnished by the purchaser, he shall have full access, at all times, to all parts of the shop where material under his inspection is being manufactured.

167. **Accepting Material or Work.**—The inspector shall stamp each piece accepted with a private mark. Any piece not so marked may be rejected at any time, and at any stage of the work. If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the purchaser.

168. **Shop Plans.**—The purchaser shall be furnished complete shop plans (§ 13).

169. **Shipping Invoices.**—Complete copies of shipping invoices shall be furnished to the purchaser with each shipment.

## FULL-SIZED TESTS.

170. **Test to Prove Workmanship.**—Full-sized tests on eye-bars and similar members, to prove the workmanship, shall be made at the manufacturer's expense, and shall be paid for by the purchaser at contract price, if the tests are satisfactory. If the tests are not satisfactory, the members represented by them will be rejected.

171. **Eye-bar Tests.**—In eye-bar tests, the fracture shall be silky, the elongation in 10 ft., including the fracture, shall be not less than 15 per cent; and the ultimate strength and true elastic limit shall be recorded (§ 141).

## ERECTION.

172. If the contractor erects the bridge he shall, unless otherwise specified, furnish all staging and falsework, erect and adjust all metal work, and shall frame and put in place all floor timbers, guard timbers, trestle timbers, etc., complete ready for traffic.

The contractor shall put in place all stone bolts and anchors for attaching the steel work to the masonry. He shall drill all the necessary holes in the masonry, and set all bolts with neat Portland cement.

173. Field rivets shall preferably be driven by pneumatic riveters of approved make. A pneumatic bucker shall be used with a pneumatic riveter. Splices and field connections shall have 50 per cent of the holes filled with bolts and drift pins (of which one-fifth shall be drift pins) before riveting. Splices and connections carrying traffic during erection shall have 75 per cent of the holes so filled. Rivets in splices of compression chords shall not be driven until the abutting surfaces have been brought into contact throughout, and submitted to full dead load stress. Field riveting shall be done to the satisfaction of the engineer.

The fence may be field bolted, all other connections shall be field riveted.

174. The erection will also include all necessary hauling from the railroad station, the unloading of the materials and their proper care until the erection is completed.

175. Whenever new structures are to replace existing ones, the latter are to be carefully taken down and removed by the contractor to some place where the material can be hauled away.

176. The contractor shall so conduct his work as not to interfere with traffic, interfere with the work of other contractors, or close any thoroughfare on land or water.

177. The contractor shall assume all risks of accidents and damages to persons and properties prior to the acceptance of the work.

178. The contractor must remove all falsework, piling and other obstructions or unsightly material produced by his operations.

## PAINTING AFTER ERECTION.

179. After the bridge is erected the metal work shall be thoroughly cleaned of mud, grease or other material, then be thoroughly and evenly painted with two coats of paint of the kind specified by the engineer, mixed with linseed oil. All recesses which may retain water, or through which water can enter, must be filled with thick paint or some waterproof cement before the final painting. The different coats of paint must be of distinctly different shades or colors, and one coat must be allowed to dry thoroughly before the second coat is applied. All painting shall be done with round brushes of the best quality obtainable on the market. The paint shall be delivered on the work in the manufacturer's original packages and be subject to inspection. If tests made by the inspector shows that the paint is adulterated, the paint will be rejected and the contractor shall pay the cost of the analyses, and shall scrape off and thoroughly clean and repaint all material that has been painted with the condemned paint. The paint shall not be thinned with anything whatsoever; in cold weather the paint may be thinned by heating under the direction of the inspector. No turpentine nor benzine shall be allowed on the work, except by the permission of the inspector, and in such quantity as he shall allow. The inspector shall be notified when any painting is to be done by the contractor, and no painting shall be done until the inspector has approved the surface to which the paint is to be applied. Paint shall not be applied out of doors in freezing, rainy, or misty weather, and all surfaces to which paint is to be applied shall be dry, clean and warm. In cool weather the paint may be thinned by heating, and this may be required by the inspector.

# APPENDIX II.

## GENERAL SPECIFICATIONS FOR CONCRETE HIGHWAY BRIDGES AND FOUNDATIONS.*

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## PART I. DESIGN.

### General Description.

1. **Classes.**—Bridges under these specifications are divided into five classes as follows:
Class A.—For city traffic.
Class $D_1$.—For country roads with heavy traffic.
Class $D_2$.—For country roads with light traffic.
Class $E_1$.—For heavy electric cars only.
Class $E_2$.—For medium electric cars only.

2. **Types.**—Concrete bridges may be divided according to design into (1) circular and box culverts; (2) slab bridges; (3) deck beam bridges; (4) through girder bridges; (5) arch bridges; (6) viaducts, and (7) trestles.

3. Types of concrete structure should preferably be selected as follows:
Box culverts up to 14 ft. span.
Slab bridges from 14 ft. to 24 ft. span.
Girder bridges from 24 ft. to 65 ft.
Arches from 6 ft. up.
Arches, except for very short spans, should not be used unless the foundations are solid rock or other materials in which settlement will not occur.

Slab and girder bridges shall be cambered one-twentieth inch per foot of span.

4. **Roadways.**—Minimum clear widths of roadway shall be provided as follows:

*Class A Bridges.*—As required by the traffic which is commonly not less than 30 feet.

*Class $D_1$ Bridges.*—For bridges and culverts with spans of 10 ft. and less, 24 ft. roadway, bridges with spans of 10 to 60 ft., 20 ft. roadway; bridges over 60 ft. span, 18 ft. roadway.

*Class $D_2$ Bridges.*—For bridges and culverts with spans of 10 ft. and less, 20 ft. roadway; bridges with spans of 10 to 60 ft., 18 ft. roadway; bridges over 60 ft. span, 16 ft. roadway.

Culverts under fills shall have a length of barrel that will give a top width of not less than 20 ft. with side slopes of 1½ horizontal to 1 vertical.

## PART II. LOADS.

5. **Dead Load.**—The dead load shall include the weight of the structure complete, including pavement and other wearing surface. In computing the dead load the following unit weights shall be used: Steel, 490 lb. per cu. ft.; concrete, plain or reinforced, 150 lb. per cu. ft.; earth fill, 100 lb. per cu. ft.; gravel, 125 lb. per cu. ft.; stone or gravel macadam, 140 lb. per cu. ft.; brick, 150 lb. per cu. ft.; granite paving, 160 lb. per cu. ft.; oak or other hard woods, 4½ lb. per board foot; pine or fir, 3½ lb. per board foot; creosoted pine or fir, 4½ lb. per board foot. The rails, splices and guard timbers for electric railways shall be assumed to weigh not less than 100 lb. per lineal foot of track.

6. **Live Loads.**—The bridges of the different classes shall be designed to carry in addition to the dead load, a moving load, either uniform or concentrated, or both, as specified below, placed so as to give maximum stresses.

7. **Class A.** *For City Traffic.*—For the floor and its supports, on any part of the roadway or on each of the street car tracks, a concentrated load of 24 tons on two axles 10 ft. centers and 5 ft. gage (assumed to occupy a width of 12 ft. for a single line and 22 ft. for a double line), and upon the remaining portion of the floor a load of 125 lb. per sq. ft., and a concentrated motor truck load as for class $D_1$. Sidewalks are to be designed for a live load of 100 lb. per sq. ft. Girders are to be designed for a load of 1,800 lb. per lineal foot on each track and also a uniform load of 125 lb. per sq. ft. on remaining floor surface.

*To accompany "General Specifications for Construction of a Highway Bridge," Chapter XXIV, page 430.

8. **Class $D_1$.** *For Country Bridges with Heavy Traffic.*—For the floor and its supports, a load of 125 lb. per sq. ft. of total floor surface or a 20-ton motor truck with axles spaced 12 ft. and wheels 6-ft. centers, with 14 tons on rear axle and 6 tons on front axle. The truck to occupy a space 10 ft. wide and 32 ft. long. The rear wheels to have a width of 20 in.

Girders are to be designed for a uniform live load of 125 lb. per sq. ft.

9. **Class $D_2$.** *For Country Roads with Light Traffic.*—For the floor and its supports, a load of 100 lb. per sq. ft. of total floor surface or a 15-ton motor truck with axles spaced 10 ft. and wheels 6-ft. centers, with 10 tons on the rear axle and 5 tons on the front axle. The truck to occupy a space 10 ft. wide and 30 ft. long. The rear wheels to have a width of 15 in.

Girders are to be designed for a uniform live load of 100 lb. per sq. ft.

10. **Class $E_1$.** *For Heavy Electric Cars Only.*—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are each loaded with a load of 40,000 lb. making a total of 160,000 lb. for each car. Or a uniform load of 6,000 lb. per lineal foot on each car track up to 50 ft., reduced to 5,000 lb. for 100 ft. and over, and proportionately for intermediate spans.

11. **Class $E_2$.** *For Medium Electric Cars Only.*—On each track a series of concentrations consisting of two pairs of trucks, the axles of the pairs being spaced 5 ft. centers, while the distance between centers of interior axles is 10 ft., the pairs of trucks being spaced 15 ft. centers. The axles are each loaded with a load of 25,000 lb. making a total of 100,000 lb. for each car, or a uniform load of 3,500 lb. per lineal foot for all spans up to 50 ft., reduced to 3,000 lb. per lineal foot for spans of 100 ft. and over, and proportionately for intermediate spans.

12. **Distribution of Concentrated Loads.**—The distribution of concentrated loads on concrete structures shall be calculated as follows:

(*a*) The distribution of concentrated wheel loads for bending moments in reinforced concrete slabs carried on longitudinal girders shall be calculated by the formula

$$e = 2(l + c)/3 \qquad (1)$$

with a maximum limit of 6 ft. for $e$, where $e$ = effective width (distance that the load may be considered as uniformly distributed on a line down the middle of the slab parallel to the supports), $l$ = span, and $c$ = width of tire of wheel, all distances in feet. See Fig. 1.

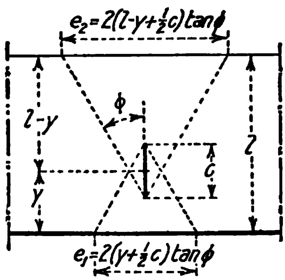



FIG. 1.

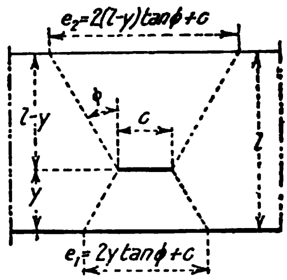



FIG. 2.

(*b*) The distribution of concentrated wheel loads for bending moments in reinforced concrete slabs with transverse girders shall be calculated by the formula

$$e = 2l/3 + c \qquad (2)$$

with a maximum limit of 6 ft. for $e$, where $e$ = effective width, $l$ = span, and $c$ = width of tire of wheel as defined in paragraph (*a*). See Fig. 2.

(*c*) The distribution of concentrated wheel loads for bending moments in slabs of girder bridges in which the span of the bridge is not less than the width of bridge center to enter of girders, shall be calculated for spans of 9 ft. or over by the formula

$$e = 2l/3 \qquad (3)$$

with a maximum limit of $e$ = 12 ft., where $e$ = effective width, and $l$ = span as defined in paragraph (*a*).

(*d*) The effective width for shear in beams carrying concentrated loads shall be taken the same as for bending moment as calculated by formula (1) or formula (2), with a minimum effective width of 3 ft. and a maximum effective width of 6 ft.

The total shear for an effective width of 3 ft. shall be considered as punching (pure) shear. The total shear for an effective width of 4.5 ft. and over shall be considered as beam shear (a measure of diagonal tension), for effective widths between 3 ft. and 4.5 ft. the total shear shall be divided proportionally between punching shear and beam shear. Beam shear shall be used in calculating bond stress and as a measure of diagonal tension.

(*e*) In the design of longitudinal joists or stringers with concrete floors, the fraction of the concentrated load carried by one stringer for spacings 6 ft. or less will be taken equal to the stringer spacing in feet divided by 6 ft.; with plank floors the fraction of the concentrated load carried by one stringer for spacings 4 ft. or less will be taken equal to the stringer spacing divided by 4 ft., the maximum in each case being the full load. Outside stringers are to be designed for the same load as interior stringers.

(*f*) In the design of transverse stringers or floorbeams with concrete floors, the fraction of the concentrated load carried by one floorbeam for floorbeams spaced 6 ft. or less, will be taken equal to the floorbeam spacing divided by 6 ft. For floorbeams spaced 6 ft. or over the entire reactions are assumed as carried by one floorbeam. Axle loads are assumed as distributed on a line 12 ft. long.

13. **Wind Load.**—Wind pressure on bridges shall be assumed at 30 lb. per sq. ft. on the greatest vertical projection of the bridge with a minimum wind load of 300 lb. per lineal foot for all classes. Bridges carrying electric cars shall be assumed to carry when loaded, a wind load of 30 lb. per sq. ft. on the greatest vertical projection of the structure and also a wind load of 400 lb. per lineal foot, applied 7 ft. above the base of rail considered as a moving load.

Trestle or viaduct towers shall be calculated for wind loads as given for bridges, and also a wind load of 100 lb. for each vertical lineal foot of trestle or viaduct bent.

Where wind stresses are added to dead and live load stresses the allowable unit stresses for dead and live loads may be increased 25 yer cent.

14. **Temperature Stresses.**—Reinforced concrete arches, frames and other restrained structures shall be designed for a variation in temperature of 80 degrees F. When temperature stresses are added to dead and live load stresses the allowable unit stresses for dead and live loads may be increased 25 per cent.

15. **Centrifugal Force of Train.**—Structures on curves shall be designed for the centrifugal force of the live load acting at the top of the rail. The centrifugal force shall be calculated by the formula

$$C = 0.03 W \cdot D$$

where $C$ = centrifugal force in pounds, $D$ = degree of curvature; and $W$ = weight of train in pounds.

16. **Longitudinal Forces.**—The effect of suddenly stopping a moving load shall be considered. The coefficient of friction of wheels sliding on rails shall be assumed as 0.2.

## PART III. UNIT STRESSES AND PROPORTION OF PARTS.

17. **Unit Stresses.**—All parts of the structure shall be proportioned so that maximum stresses shall not exceed the following:

18. **Impact.**—(*a*) For concrete arches with spandrel filling or culverts with a minimum filling of one foot, no allowance for impact.

(*b*) For concrete slab and girder bridges and trestles and arches without spandrel filling, 30 per cent for impact.

(*c*) For steel bridges the following allowance for impact.

For the floor and its supports including floor slabs, floor joist, floorbeams and hangers, 30 per cent.

For all truss members other than the floor and its supports, the impact increment shall be $I = 100/(L + 300)$, where $L$ = length of span for simple highway spans (for trestle bents, towers, movable bridges, arch and cantilever bridges, and for bridges carrying electric trains, $L$ shall be taken as the loaded length of the bridge in feet producing maximum stress in the member).

Impact shall not be added to stresses produced by longitudinal, centrifugal, lateral or wind forces, or temperature stresses.

19. **Calculation of Stresses.**—The following assumptions are to be used as a basis for calculation.

1. Calculations are to be made with reference to working stresses and safe loads.
2. A plane section before bending remains plane after bending.
3. The modulus of elasticity of concrete in compression is constant, and the distribution of stresses in beams is rectilinear.
4. In calculating the moment of resistance in beams the tensile stresses in the concrete are neglected.

5. Adhesion between concrete and reinforcing steel is assumed as perfect, the two materials being assumed as stressed in proportion to their moduli of elasticity.

6. The ratio of the modulus of elasticity of steel to the modulus of elasticity of concrete is taken as 15.

7. Initial stress in reinforcement due to contraction of the concrete is neglected.

20. **Lengths of Span.**—The span length of girders, beams, and slabs simply supported shall be taken as the distance from center to center of supports, but need not be taken to exceed the clear span plus the depth of beam or slab. For continuous or restrained beams the span length may be taken as the clear distance between faces of supports. Where monolithic brackets are used the face of the support may be taken as that point where the combined depth of beam and bracket are one-third greater than the depth of beam at the center of the span. The reduction in the length of span where brackets are used shall in no case be more than one-tenth the span where a bracket is used at one end, or two-tenths the span where brackets are used at both ends of the beam or girder. Maximum negative moments are assumed as existing at the end of the span as defined above.

For calculating stresses, the span of a concrete arch shall be taken as the span of the neutral axis of the arch ring; and the rise of the arch shall be taken as the distance from the line connecting the ends of the neutral axis to the neutral axis at the crown.

The actual span and rise of an arch shall be taken as the clear distance between springing lines and the clear rise to the intrados of the arch.

21. **Bending Moments.**—For simple beams the moments due to external loads shall be calculated by the usual methods; for partially continuous beams the maximum positive bending moment near the center and the maximum negative bending moment at the end of the beam shall be taken as $\frac{8}{10}$ the maximum positive moment in a simple beam; for continuous beams the maximum positive bending moment at or near the center of the beam and the maximum negative bending moment at the end of the span shall be taken at $\frac{8}{12}$ the maximum positive moment in a simple beam.

For spans of unusual or unequal length, or spans carrying heavy concentrated loads more exact calculations shall be made.

22. **T-beams.**—In beam and slab construction an effective bond shall be provided at the junction of the beam and the slab. When the principal slab reinforcement is parallel to the beam, transverse reinforcement shall be used extending over the beam and well into the slab.

The slab may be considered an integral part of the beam when adequate bond and shearing resistance between slab and web of beam is provided, but its effective width shall be determined by the following rules:

(*a*) It shall not exceed one-fourth of the span length of the beam.

(*b*) Its overhanging width on either side of the web shall not exceed six times the thickness of the slab.

T.-beams used mainly for the purpose of providing additional compression area of concrete shall have a width of flange not more than three times the width of the stem, and a thickness of flange of not less than one-third the depth of the beam.

23. **Floor Slabs Supported on Four Sides.**—Floor slabs supported on four supports shall be continuous over the supports. For square slabs one-half the load shall be assumed as carried by the reinforcement in each direction. For oblong slabs in which the length of slab is not greater than 1.5 times its width the proportion of the load taken by the transverse reinforcement shall be assumed as $r = l/b - 0.5$, where $l$ = length and $b$ = width of slab. The remainder of the load is to be taken by the longitudinal reinforcement. Where $l$ is equal or greater than $1\frac{1}{2}$ times $b$ all the load shall be assumed as taken by the transverse reinforcement.

In placing the reinforcement two-thirds of the previously calculated bending moments shall be assumed as taken by the center half of the slab and one-third by the outside quarters.

24. **Bond Strength.**—Adequate bond strength shall be provided. In restrained and cantilever beams the reinforcing bars shall be anchored in the support sufficiently to develop the full tensile strength of the bars. Where high bond strength is required, deformed bars may be used, or the bond strength may be increased by using hooked ends on reinforcing bars. Hooked ends on reinforcing bars shall consist of turns through 180 degrees.

25. **Spacing of Reinforcement.**—The lateral spacing of parallel reinforcing bars shall not be less than three diameters from center to center, nor shall the distance from the side of the beam to the center of the nearest bar be less than two diameters. The clear spacing between two layers of bars shall be not less than 1 inch. Where more than two layers of bars are used the layers shall be tied together by adequate metal connections at and near points where bars are bent up. Where more than one layer of bars is used at least all of the bars above the lower layer should be bent up and anchored beyond the edge of the support.

26. **Shear Reinforcement.**—Two-thirds of the external vertical shear shall be taken as producing stresses in web reinforcement. Vertical or inclined stirrups shall be secured to the horizontal members in such a way as to prevent slip. Sufficient bond resistance between stirrups or diagonals shall be provided in the compression area of the beam. The longitudinal spacing of vertical stirrups shall not exceed one-half the depth of the beam, and inclined web members shall be spaced not to exceed three-fourths of the depth of the beam. Where horizontal bars are bent up to carry web stresses the points of bending up shall not be spaced to exceed three-fourths of the depth of the beam. In restrained beams the first stirrup or place of bending down of bar shall be placed not further than one-half the depth of the beam from the face of the support.

When a flat slab rests on a column or a column bears on a footing, or a concentrated load is applied near the end of a short beam, the shear in the slab shall be considered as punching shear.

27. **Columns.**—Columns shall preferably not be greater in length than 15 times the least width. Columns shall be reinforced by both vertical reinforcing bars and bands, hoops or spirals, or by structural shapes so arranged as to enclose the concrete core. The effective area of hooped columns or columns reinforced with structural shapes shall be taken as the area within the circle enclosing the spiral or the polygon enclosing the structural shapes.

The minimum size of columns shall be 12 in. out to out.

Longitudinal reinforcement shall be assumed to carry its proportion of stress. Hoops or bands shall not be counted on as carrying stress.

Hooping shall have a clear spacing not greater than one-sixth the diameter of the enclosed column, and preferably not greater than one-tenth and in no case more than 2½ inches. Hooping is to be circular and the ends of the bands shall be united so as to develop their full strength. Hooping shall not be less than 1 per cent of the enclosed column. Bending stresses in columns due to eccentric loads, or due to lateral forces shall be provided for by increasing the section until the maximum stresses do not exceed the permissible values.

28. **Temperature Stresses.**—Temperature reinforcement shall not be less than one-third of one per cent of gross area, and of a form that will develop high bond resistance, placed near the exposed surface and well distributed.

29. **Expansion Rockers.**—Reinforced concrete bridges with spans of 40 ft. or over shall be provided with expansion rockers on one end of each span. Rockers shall be made of cast-iron that will comply with the specifications of the American Society for Testing Materials for gray iron castings. Rockers shall have a thickness of not less than 2½ in. for spans of 45 ft. and less, and not less than 3 in. for spans greater than 45 ft. The upper and lower edges of the rocker shall be turned to a radius equal to one-half the height of the rocker. The bearing of the rocker on steel bearing plates shall not exceed $300d$ lb. per sq. in., where $d$ = height of rocker.

30. **Bearing Plates.**—The rockers shall turn between steel bearing plates with planed bearing surfaces. The cut of the tool shall be in the direction of expansion. The bearing of the steel plates on the concrete shall not exceed 600 lb. per sq. in. The bending stress in the steel bearing plates shall not exceed 16,000 lb. per sq. in. The bearing plates shall be set in full mortar beds and accurately leveled.

Bearing plates without rockers shall be used on one end of girder bridges with spans of less than 40 ft.

31. **Rocker Pockets.**—Rocker pockets two inches longer than the rockers shall be provided in the concrete. The top of the rocker shall come ½ in. above the surface of concrete.

The rockers shall be placed accurately at right angles to the axis of the girder and shall be supported in position. Rockers may be supported by soft wooden struts not more than one inch square which have been soaked with water previous to driving. The pocket shall then be filled with asphalt. The top plates shall then be placed in position and held in level position by soft wooden sticks not more than one inch square placed vertically one on each side of the rocker and resting on the bottom of the pocket. The bituminous felt cushion used to separate the superstructure concrete from the substructure shall not over lap the steel plate more than one inch. No concrete shall be permitted to enter the rocker pocket. The asphalt used shall comply with the specifications of the American Society for Testing Materials for asphalt for waterproofing.

Bituminous felt shall be provided in sheets not less than ½ in. thick. Ordinary tar or building paper shall not be used.

32. **Specifications for Concrete Floors.**—Concrete floors shall be built of 1–2–4 Portland cement concrete. The distribution of loads shall be as given in § 12. All concrete floors not covered with a bituminous wearing surface are to be finished with a wearing surface ½ in. thick of 1 to 1 mortar. This mortar coat is to be applied immediately after the slab is poured and the surface shall be rubbed with a heavy wood float to give a smooth surface.

Floors shall be concreted in a continuous operation over each span. Expansion joints shall be provided between spans and at the ends of floors resting on abutments. These joints shall be filled with tar or asphalt or tar or asphalt felt as shown on the drawings. At expansion joints the

edges of the concrete shall be protected by steel plates and the joints filled with tar or asphalt felt. Steel spans shall be swung clear of the false-work before concrete is poured on the floor. On long spans the floors shall be poured on both ends at the same time. Concrete slabs shall be protected from the direct rays of the sun and the surface shall be kept moist for a minimum of one week.

33. **Bituminous Floor Coating.**—A tar or asphalt as specified by the engineer shall be applied hot to the concrete at the rate of one-third gallon per square yard. Over this coating while hot shall be sifted hot, clean, dry sand screened through a ¼-in. mesh screen. The sand shall be placed in excess and rolled with a hand roller. All joints and corners shall be filled with asphalt or tar. All concrete surfaces to be covered with bituminous coating shall be thoroughly cleaned with steel brooms and swept clean.

## PART IV. WORKING STRESSES.

34. The following working stresses are for static loads and are based on a concrete composed of one part Portland cement, two parts sand or fine aggregate, and four parts stone or coarse aggregate, that will develop an ultimate compressive strength of 2,000 lb. per sq. in. at an age of 28 days in cylinders 8 in. in diameter and 16 in. long, when made and stored in moist air under laboratory conditions. Concretes of different mixtures shall have allowable stresses proportional to their ultimate compressive strengths determined under the above conditions.

35. **Bearing.**—When compression is applied to a surface of concrete having at least twice the loaded area, a stress of 700 lb. per sq. in. may be allowed on the area actually under load.

36. **Axial Compression.**—For concentric compression on a plain concrete pier, the length of which does not exceed four diameters, a stress of 450 lb. per sq. in. may be allowed.

37. **Columns.**—Reinforced concrete columns may have allowable stresses as follows:

(*a*) Columns with longitudinal reinforcement of not less than 1 per cent and not more than 4 per cent, and with lateral ties of not less than ¼ in. in diameter, 12 in. apart, nor more than 15 diameters of longitudinal bar, may have an allowable stress in the concrete of 450 lb. per sq. in.

(*b*) Columns with longitudinal reinforcement of not less than 1 per cent and not more than 4 per cent, and with circular hoops or spirals not less than 1 per cent of the volume of the concrete, where the hoops are spaced not more than one-sixth the diameter of the enlcosed column or more than 2½ in., may have an allowable stress in the concrete of 700 lb. per sq. in.

38. **Compression on Extreme Fiber.**—The extreme fiber stress of a beam, calculated on the assumption of a constant modulus of elasticity of concrete shall not exceed 650 lb. per sq. in. Adjacent to the support of continuous beams, stresses 15 per cent higher may be used.

39. **Shear and Diagonal Tension.**—As a measure of the diagonal tension the following allowable shearing stresses may be used.

(*a*) For beams with horizontal bars only and without web reinforcement, a shearing stress of 40 lb. per sq. in. may be allowed.

(*b*) For beams with web reinforcement of stirrups looped about longitudinal reinforcing bars in the tension side of the beam and spaced horizontally not more than one-half the depth of the beam or for beams in which longitudinal bars are bent-up at an angle of not more than 45 degrees or less than 20 degrees with the axis of the beam, and the points of bending are spaced horizontally not more than three-quarters the depth of the beam, a shearing stress of 90 lb. per sq. in. may be allowed.

(*c*) For a combination of bent bars and vertical stirrups looped about reinforcing bars on the tension side of the beam and spaced horizontally not more than one-half the depth of the beam, a shearing stress of 100 lb. per sq. in. may be allowed.

(*d*) For beams with web reinforcement (either vertical or inclined) securely attached to the longitudinal bars on the tension side of the beam in such a way as to prevent slipping of bar past the stirrups, vertical stirrups to be spaced not more than one-half and inclined members not more than three-fourths the depth of the beam, either with longitudinal bars bent up or not, a shearing stress of 120 lb. per sq. in. may be allowed.

(*e*) For punching or pure shear, 120 lb. per sq. in. may be allowed.

In calculating the stresses in web reinforcement two-thirds of the external vertical shear is to be assumed as taken by the stirrups or bent-up bars. The stresses in stirrups when combined with bent-up bars are to be determined by finding the total amount of shear that may be allowed by reason of the bent-up bars, and subtracting this shear from the total external vertical shear. Two-thirds of the remainder will be the shear to be carried by the stirrups.

40. **Bond Stress.**—The bond stress between concrete and plain reinforcing bars may be assumed at 80 lb. per sq. in.; on drawn wire 40 lb. per sq. in.; on deformed bars 100 lb. per sq. in. Bars with ends hooked by bending through 180 degrees around a radius of two diameters of bar may have a bond stress of 120 lb. per sq. in.

41. **Stresses in Steel Reinforcement.**—The tensile or compressive stress in steel reinforcement shall not exceed 16,000 lb. per sq. in. The tensile stress in stirrups shall not exceed 12,000 lb. per sq. in.

## PART V. CONCRETE ARCHES.

42. **Proportions.**—The crown thickness of reinforced arches shall not be less than one-sixtieth of the span, nor the thickness at the springing line less than twice the thickness at the crown.

43. **Reinforcement.**—Arch reinforcement shall be double and preferably symmetrical. There shall be sufficient steel at every section to take all the tension on the assumption that the concrete takes no tension. The main reinforcing bars shall be fastened together by means of stirrups not less than ⅜ inches in diameter, and spaced not more than the depth of the arch ring at the spring. The stirrups shall pass around the main bars and shall be rigidly wired in place. The transverse reinforcement shall be not less than one-third of one per cent and shall be symmetrically placed on each side of the arch ring. Reinforcing steel shall be placed and secured in position before the concrete is deposited. The bars shall be blocked up from the forms by means of concrete blocks or metal chairs. The concrete covering shall not be less than one inch. The area of steel at the crown shall not be less than one per cent.

44. **Analysis of Stresses.**—Arches shall be analyzed by the elastic theory for at least the following live load conditions.

1. The live load covering the middle quarter of the span for maximum positive bending moment at the crown.
2. The live load covering three-eighths the span on each end, or a total of three-fourths the span for maximum negative bending moment at the crown.
3. The live load covering five-eighths of the span loaded from one end for maximum positive moment at the spring on the unloaded side.
4. The live load covering three-eighths of the span for maximum negative moment at the spring on the loaded side.

Arches with unsymmetrical spans or of unusual design or loads shall be investigated by means of influence lines.

45. Arch ribs shall be designed for a variation in temperature of 40 degrees F. on each side of the mean.

46. The effect of the shortening of the arch ring due to axial compression shall be considered.

47. The spandrel walls shall be securely anchored to the arch ring, which shall be reinforced transversely to provide for the maximum lateral thrust on the spandrel walls.

48. The top of the arch ring and spandrel walls covered with earth shall be given a smooth coating with cement mortar, and shall be waterproofed with neat cement grout or with bituminous coating.

49. Expansion joints shall be left in spandrels and handrail. The minimum number of expansion joints shall be three for spans less than 50 ft. and five for spans over 50 ft. Expansion joints shall be of tongue and groove type thoroughly waterproofed.

## PART VI. MATERIALS.

50. **Portland Cement.**—The cement shall be Portland cement of an approved brand, and shall conform to the standard specifications of the American Society for Testing Materials, effective January 1, 1917. Tests shall be made from each car load of cement. Cement shall be delivered so as to give not less than 10 days for testing before it is used. Cement shall be stored so that it will not be damaged by moisture.

51. **Water.**—The water used in mixing concrete shall be clean and fresh and free from oil, acid, alkali, or organic matter.

52. **Sand.**—The sand or fine aggregate shall consist of clean siliceous grains uniformly graded in size, from fine to coarse and passing when dry through a screen having ¼ in. diameter holes. Not more than 20 per cent by weight shall pass a sieve having 50 meshes per lineal inch, nor more than 6 per cent pass a sieve with 100 meshes per lineal inch. Sand shall not contain more than 3 per cent of clay by actual dry weight, and shall be free from soft particles, lumps of clay, vegetable loam or other organic matter.

The sand shall be of such a quality that a mortar composed of one (1) part Portland cement and three (3) parts sand by weight when made into briquettes shall show as high tensile strength, at an age of 7 days as 1–3 mortar of the same consistency made with the same cement and standard Ottawa sand.

53. **Broken Stone or Gravel.**—Coarse aggregate shall consist of crushed stone or gravel which is retained on a screen having ¼ in. diameter holes and all passing a screen as follows:

For concrete for heavy foundations, all stone shall pass a 2½-in. screen, and at least 50 per cent shall be retained on a ¾-in. screen.

For concrete used for slabs, columns, arch rings and similar structures, all stone shall pass a 1½-in. screen, and at least 50 per cent shall be retained on a ½-in. screen.

For concrete for very thin sections, all stone shall pass a 1-in. screen, and not less than 40 per cent shall be retained on a ½-in. screen.

All stone or gravel shall consist of clean, hard durable material, and shall be graded from coarse to fine. Stone dust or dirt adhering to the particles as a film shall be removed by washing or the material shall be rejected. Any material containing shale, lumps of clay, disintegrated or rotten boulders, or clay exceeding 3 per cent dry weight shall be rejected. The engineer may permit the use of unscreened gravel subject to frequent tests to determine the relative proportions of sand and gravel. In all cases where proportions of concrete are stated the sand and gravel or broken stone are to be measured separately.

54. **Reinforcing Steel.**—All reinforcing steel shall be made by the open-hearth process, and shall comply with the requirements for billet steel, reinforcement bars adopted by the American Society for Testing Materials, revised 1914. Unless otherwise shown on the drawings reinforcing steel shall be plain round or square bars of the structural steel grade. Deformed bars shall be of a type approved by the engineer. Twisted bars will not be accepted under these specifications. All bars shall be free from rust, dirt, paint or grease when placed in the work. All reinforcing steel shall in general be wired or otherwise held rigidly in position before the concrete is deposited.

All structural steel used for concrete reinforcement shall comply with the requirements for steel as given in the author's Specifications for Steel Highway Bridges.

## PART VII. DETAILS OF CONSTRUCTION.

55. **Proportions.**—The materials shall be carefully selected, of uniform quality, and proportioned with a view to securing as nearly as possible a maximum density, which is obtained by grading the aggregates so that the smaller particles fill the spaces between the larger thus reducing the voids in the aggregate to a minimum. Sand and broken stone shall be measured by loose volume. A bag of cement containing 94 lb. net weight shall be assumed as the equivalent of one cubic foot.

56. Sand and broken stone or gravel shall be used in such proportions as to produce maximum density. The proportions shall be carefully determined by density experiments, and the grading of the fine and coarse aggregate shall be uniformly maintained, or the proportions changed to meet the varying sizes.

57. For reinforced concrete slabs, beams, columns, etc., the proportions shall generally be one (1) part Portland Cement, two (2) parts sand and four (4) parts broken stone or gravel. For foundations plain or reinforced the proportions shall generally be one (1) part Portland cement, two and one-half (2½) parts sand and five (5) parts broken stone or gravel. For thin sections or columns or members requiring a stronger concrete the proportions shall generally be one (1) part Portland cement, two (2) parts sand and three (3) parts broken stone or gravel.

58. The proportions in every case shall be determined by the strength or other qualities required in construction. Advance tests shall be made on concrete composed of materials to be used in the work.

59. **Mixing.**—The proportions of the various ingredients shall be determined by accurate methods of measurement. The concrete shall be thoroughly mixed in a mixer of a type which will insure the uniform distribution of materials throughout the mass, and shall continue for a minimum time of one and one-half minutes after all materials have been assembled in the mixer. For mixers of two or more yards capacity the minimum time of mixing shall be two minutes. The time of mixing shall preferably be longer than the above minimum. The mixer should be equipped with a device for automatically locking the discharging device so as to prevent the emptying of the mixer until all the ingredients have been mixed for the minimum time. The water shall be measured into the mixer. The mixer shall be operated so as to give at the periphery of the drum a uniform speed of about 200 feet per minute.

60. Hand mixing shall be done on a watertight platform and especial precautions taken after the water has been added to turn all the ingredients together at least six times, and until the mass is homogeneous in appearance and color.

61. The materials shall be mixed wet enough to produce a concrete of such a consistency as will flow sluggishly into the forms and about the metal reinforcement, and which at the same time, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar. The quantity of water is of the greatest importance in securing concrete of maximum strength and density; too much water is as objectionable as too little.

62. The remixing of mortar or concrete that has partly set shall not be permitted.

63. **Placing Concrete.**—Concrete shall be conveyed rapidly from the mixer to the forms, and under no circumstances shall concrete be used that has partly set. Concrete shall be deposited in such a manner as will permit the most thorough compacting, such as can be obtained by working with a straight shovel or slicing tool kept moving up and down until all the ingredients are in

their proper place. Special care shall be taken to prevent the formation of laitance; where laitance has formed it shall be removed.

64. Reinforcing steel shall be carefully placed in accordance with the plans, and shall be held in position until the concrete is deposited and compacted. Forms shall be substantial, free from debris, and shall be thoroughly wetted or oiled. When the placing of concrete is suspended, all necessary grooves for forming future work shall be made before the concrete has set. When work is resumed, concrete previously placed shall be roughened, cleansed of foreign material and laitance, thoroughly wetted and then slushed with 1–2 Portland cement mortar. The surface of concrete exposed to premature drying shall be kept covered and wet for a period of seven days.

65. Where concrete is conveyed by spouting, the plant shall be of such a size and design as to ensure practically a continuous stream in the spout. The angle of the spout with the horizontal should be such as to allow the concrete to flow without a separation of the ingredients, in general an angle of 27 degrees with the horizontal is good practice. The spout shall be thoroughly flushed with water before and after each run. Where the delivery is intermittent, a hopper shall be provided at the bottom. Spouting through a vertical pipe is satisfactory where the flow is continuous, but where the flow is not continuous the flow shall be checked by baffle plates.

66. **Freezing Weather.**—Concrete shall not be mixed or deposited at a freezing temperature, unless special precautions are taken to prevent the use of materials covered with ice crystals, or containing frost, and to prevent the concrete from freezing before it has set and sufficiently hardened.

When the temperature of the air is below 40° F. during the time of mixing and placing concrete, the water used in mixing the concrete shall be heated to such a temperature that the temperature of the concrete mixture shall not be less than 60° F. when it reaches its final position in the forms. Care shall be used that the cement shall not be injured by boiling water. The use of salt to lower the freezing point of concrete will not be permitted.

67. **Rubble Concrete.**—Where the concrete is to be deposited in massive work, clean, large stones, evenly distributed, thoroughly bedded and entirely surrounded by concrete may be used, at the option of the engineer.

68. **Forms.**—Forms shall be substantial and unyielding and built so that the concrete shall conform to the designed dimensions and contours, and so constructed as to prevent the leakage of mortar.

For all important work the lumber used for face work shall be dressed to uniform thickness and width, shall be sound and free from loose knots, and shall be secured to the studding or uprights in horizontal lines. For backing and other rough work undressed lumber may be used. Square corners shall be filleted to give round or beveled corners as required.

Lumber used the second time shall be cleaned, and resized if necessary to insure plane surfaces.

Forms shall not be removed until authorized by the engineer. The forms on box culverts 8 ft. by 8 ft. and slab, girder and arch bridges shall remain in place in warm weather not less than three (3) weeks, and in cold weather at discretion of engineer.

69. **Joints.**—Concrete structures shall wherever possible be cast in one operation, but when this is not possible, the resulting joint shall be formed where it will least impair the strength and appearance of the structure.

Concrete in abutments shall be placed in uniform layers across the length of the abutment, care being taken to secure a good bond between the footings and the abutment wall. A full longitudinal section of the floor slab shall be run continuously, any unavoidable joints being made in a longitudinal direction. Each girder shall be concreted in a continuous operation, the concrete being placed in layers along the length of the girder.

70. Concrete in arch rings shall be deposited continuously in one operation. If the entire arch ring cannot be deposited continuously in ten (10) hours, the arch ring shall be divided into transverse segments, each of which is to be deposited in a continuous operation.

71. Joints in columns shall be made flush with the lower side of girders. Joints in beams and slabs shall come at or near the center of the span.

Joints in columns shall be perpendicular to the axis, and in girders, beams and floor slabs, perpendicular to the plane of their surfaces. Joints in arch rings shall be on radial planes.

72. Before placing concrete on top of a freshly poured column at least two hours shall be allowed for settlement and shrinkage.

73. In massive retaining walls and abutments built without reinforcement, expansion joints shall be provided at distances apart equal to one and one-half times height of the wall.

74. **Reinforcement.**—The length of lap shall be determined on the basis of safe bond stress, the stress in the bar and the shearing resistance of the concrete at the point of splice, or a connection shall be made between the bars of sufficient strength to carry the stress. Longitudinal tension bars in all slab bridges shall be furnished full length as specified on the plans. Shop or

field splicing of longitudinal tension bars in girders will not be permitted within the middle half of the span length. There shall not be more than one splice per bar, and adjacent bars shall not be spliced at the same end of girder. All slabs shall be well reinforced transversely as well as longitudinally. Stirrups shall be used in all girder bridges to carry shear. At foundations bearing plates shall be provided for supporting the bars, or the bars may be carried into the footing a sufficient distance to transmit the stress in the steel to the concrete by means of the bearing and bond resistance.

75. **Waterproofing.**—Concrete proportioned to obtain the greatest practicable density and mixed to the proper consistency will be impervious under moderate pressures. Thin concrete walls in direct contact with the earth shall be reinforced with horizontal and vertical reinforcement placed near the surface in contact. For this purpose one-third of one per cent of reinforcement in each direction is satisfactory. For walls in direct contact with the earth the application of one or two coatings of hot coal tar pitch, following a painting with a thin wash of coal tar dissolved in benzol to the thoroughly dried surface of the concrete, gives excellent results. A coal tar paint made by mixing 16 parts refined coal tar, 4 parts Portland cement, and 3 parts kerosene oil is an excellent waterproofing paint.

76. **Surface Finish.**—As soon as the forms are removed all rough surfaces shall be filled with mortar of the same grade as the concrete used, and if necessary to insure a uniform appearance and smoothness, the entire surface of the structure shall be thoroughly rubbed with carborundum or other abrasive material, and all uneven or irregular portions removed.

## PART VIII. SUBSTRUCTURES AND FOUNDATIONS.

### Concrete Abutments, Piers and Retaining Walls.

77. **Type.**—The design may be of the reinforced concrete cantilever or counterfort section, or of gravity section, capable of resisting the overturning action of the earth and the impact of ice jams or floating debris. In all types of design, steel reinforcing shall be used at points where tension may be developed.

78. **Data for Design.**—The thrust of the filling on the wall shall be calculated by Rankine's theory, using an angle of repose of 1½ to 1 for earth filling and 1 to 1 for sand filling. The actual weight of the filling shall be used, with a minimum of 100 lb. per cu. ft.

The weight of piers and abutments shall be reduced for buoyancy of water where the foundation material is saturated.

The pressure of a flowing stream on a pier shall be taken as follows:

For square piers

$$P = 1.24a \cdot v^2 \tag{1}$$

For round piers

$$P = 0.62a \cdot v^2 \tag{2}$$

For piers three times as long as broad

$$P = 1.30a \cdot v^2 \tag{3}$$

For a pier five or six times as long as broad and with a cutwater, having plane faces and an angle of 30 degrees between cutwater faces

$$P = 0.46a \cdot v^2 \tag{4}$$

where $P$ = total pressure on pier, $a$ = wetted surface normal to current in square feet, $v$ = velocity of current in feet per second.

79. **Allowable Stresses.**—The allowable stresses in concrete shall be taken the same as for reinforced concrete bridges.

80. **Allowable Bearing on Foundations.**—The safe bearing on soils shall be determined by actual tests. The loads on foundations shall not exceed the following in tons per sq. ft.:

| | |
|---|---|
| Ordinary clay and dry sand mixed with clay | 2 |
| Dry sand and dry clay | 3 |
| Hard clay and firm coarse sand | 4 |
| Firm coarse sand and gravel | 5 |
| Shale rock | 8 |
| Hard rock | 20 |

For all soils inferior to above never more than one (1) ton per sq. ft.

81. **Piling.**—Foundations shall be carried to a depth sufficient to insure protection against scour and to secure satisfactory bearing. If satisfactory bearing cannot be secured foundation piling shall be used.

Piling shall be spaced from 2 ft. 6 in. to 3 ft. centers. The minimum distance from the center of pile to edge of footing shall be fifteen (15) inches.

Timber piling shall be white, post or burr oak, long-leaf pine, Douglas fir, cedar, cypress, chestnut or redwood. Piles shall be cut from sound trees, close grained and solid, and shall be free from defects such as injurious ring shakes, large and unsound or loose knots, decay or other defects which may materially impair their strength or durability. Piles shall be cut above the ground swell, and shall have a uniform taper from butt to tip. Short bends will not be allowed. A line drawn from the center of the butt to the center of the tip shall be within the body of the pile. Unless otherwise allowed piles shall be cut when the sap is down. Piles shall be peeled and all knots trimmed close to the body of the pile. Piles shall have a diameter of not less than eight (8) inches at the tip, and ten (10) inches at the butt. The tops of piles shall not extend above low water in the stream unless the piles are creosoted.

82. **Safe Loads on Piles.**—The allowable safe load on piles driven with a drop hammer shall be calculated by the formula

$$P = \frac{2W \cdot h}{s + 1} \tag{5}$$

where $P$ = safe load in pounds, $W$ = weight of hammer in pounds, $h$ = free fall of hammer in feet, and $s$ = average penetration of last six blows in inches. If head of pile becomes broomed in driving, the top shall be cut to sound wood before making the test. For a steam hammer substitute $\frac{1}{10}$ in place of unity in denominator in above formula.

The minimum projection of piles into the concrete shall be twelve (12) inches.

The minimum penetration of piles in foundations shall be 10 ft.

83. **Footings.**—The footings shall extend a minimum distance of four (4) feet below bed of stream unless solid rock is encountered at a shallower depth. Before any concrete is placed in footings the engineer shall approve the depth and character of the foundation.

84. **Reinforcement.**—Reinforcement shall be placed and secured in position before the concrete is poured.

85. **Concrete.**—For reinforced concrete abutments, piers and wing walls and retaining walls a 1–2–4 Portland cement concrete shall be used.

For abutments, piers and retaining walls without reinforcement a 1–2½–5 Portland cement concrete may be used.

86. **Coffer Dam.**—The inside dimensions of the coffer dam shall be sufficiently large to give easy access to all parts of the foundation forms. Concrete shall not be placed in running water, and shall only be placed in still water with suitable appliances and under direction of the engineer.

87. **Rock Anchorage.**—Where rock foundations are found, the footings shall be carried not less than six (6) inches into the rock, and the concrete shall be anchored to the rock by means of steel anchors.

88. **Ice Breakers.**—On piers built in streams carrying heavy ice, an ice breaker with not less than an 8 in. × 8 in. × ½ in. angle for cutting edge, shall be provided and anchored with bolts as shown on the plans.

89. **Drainage.**—Adequate drainage for backs of abutments and retaining walls shall be provided.

90. **Bed Plates.**—Bed plates shall be set accurately in place and imbedded in 1–2 Portland cement mortar, which shall be allowed to set for at least 48 hours before being loaded.

## SPECIFICATIONS FOR STEEL TUBULAR PIERS FOR HIGHWAY AND ELECTRIC RAILWAY BRIDGES.

91. **Thickness of Plates.**—The plates for the tubes shall be not less than ¼ in. thick for tubes up to 30 in. in diameter, not less than $\frac{5}{16}$ in. for tubes from 30 to 48 in. in diameter, and not less than ⅜ in. for tubes from 48 to 72 in. in diameter. Where the plates are in contact with the soil the thickness shall be increased at least $\frac{1}{16}$ in. For $\frac{5}{16}$ in. plate and less use ⅝ in. rivets; for ⅜ in. plate and over use ¾ in. rivets.

92. **Riveting.**—The horizontal seams shall be single lap joints riveted with a pitch of 4 diameters of rivet, while the vertical seams shall preferably be butt riveted with single riveting spaced 4 diameters of rivet, up to 48 in. diameter of tubes, and double riveting with 3 in. spacing for tubes of larger diameter.

93. **Bracing.**—The bracing between cylinders shall be a solid web below high water level. Above high water level the bracing may consist of struts and diagonal rods. The diagonal rods in open bracing shall be inclined at an angle with the vertical of not less than 45 degrees. The rods shall be upset and be provided with turn buckles. The bracing shall be made sufficiently

strong to maintain the cylinders in an upright position when acted upon by the prescribed lateral wind loads, without assistance of piles.

94. **Piles.**—Piers 30 in. or less in diameter shall have one pile, and one additional pile shall be added for each increase of six (6) inches in diameter of cylinder. A cylinder 72 in. in diameter will then have eight (8) piles.

95. **Materials and Workmanship.**—The materials and workmanship shall comply with the specifications for the highway bridge superstructure.

96. **Erection.**—Where the bottom will permit, the tubes shall be sunk well below possible scour by loading the tube and excavating the material from the inside. For this purpose a clam-shell bucket is very effective. Driving the tube with a pile driver will cut off the rivets in the horizontal seams and will not be permitted. After the tube is sunk, piles are to be driven inside of the steel shell, as closely together as possible, using care to get no pile nearer than 4 to 6 in. to the steel shell. The piles shall be driven to a good refusal; and the tops sawed off below the low water mark and reaching at least 2 diameters of the tube above the bottom. The space inside the tubes shall then be filled with concrete well tamped. Concrete shall not be deposited in running water if possible to prevent it.

Where piers are founded on rock, the tubes are to be anchored to the rock and then filled with concrete. Or cribs may be sunk on the rock and the tube set in a pocket in the crib and resting on the rock. The space outside the tube is then filled with concrete and the tube is filled with concrete in the usual manner.

## SPECIFICATIONS FOR STONE MASONRY.

1. **Quality of Masonry.**—The masonry shall consist of pitch-faced squared-stone masonry, laid in courses not less than twelve inches (12") thick, decreasing regularly from bottom to top of wall.

2. **Quality of Stone.**—The stone shall be clean, hard and of a kind known to be durable, subject to the approval of the engineer.

3. **Size of Stones.**—Stones, except for filling joints shall have a thickness not less than 12 inches, nor less than 15 inches in width, nor less in width than in thickness. Stretchers shall not be less than 2½ feet long, nor less than 1½ times the width. Headers shall go entirely through the wall where the wall is 4 ft. thick or less.

4. **Dressing.**—The top and bottom beds shall be approximately parallel to each other and to the natural bed, and shall be dressed to a surface that will admit of laying with vertical and horizontal joints not to exceed 1 inch, for a distance of 10 inches from the face. Corner stones shall have a chisel draft one and one-half inches wide. Faces shall be pitched to true lines.

5. **Laying.**—All stones shall be thoroughly drenched and laid in full mortar beds, and all vertical joints shall be completely filled with mortar for a depth of 12 inches from the face and mortar and spalls for the remainder of the vertical joint. No spalls shall be used in bed joints. One header shall be used to each three stretchers. Joints shall be broken at least 9 inches. Backing shall be carried up level with the face stones, and shall be laid with full mortar beds and joints, with joints broken at least 6 inches. Heavy hammering will not be allowed on wall after a course is laid. Stone becoming loose after mortar is set shall be relaid with fresh mortar. Stone shall not be laid in freezing weather unless directed by the engineer. If laid in freezing weather the stone shall be freed from ice, snow and frost by heating; the sand and water used in the mortar shall be heated.

6. **Mortar.**—The mortar shall be composed of one (1) part Portland cement and two and one-half (2½) parts sand, the cement and sand to be of the quality specified for concrete.

7. **Coping.**—Coping stones shall extend the full width of the wall with a 6 inch projection on each face.

8. **Pointing.**—After the wall is completed all face joints shall be raked out to a depth of one and one-half (1½) inches, dampened and bead pointed with a 1 to 2 Portland cement mortar.

# APPENDIX III.

## STRUCTURAL TABLES.

The following structural tables have been taken from the author's "Structural Engineers' Handbook." Only those tables have been included in this book that are not available in the Carnegie or Cambria steel books.

### STRUCTURAL TABLES.

## TABLE 1.

### Areas of Angles

Areas in Square Inches
Dimensions in Inches

**Angles with Equal Legs**

| Size | 1/8 | 3/16 | 1/4 | 5/16 | 3/8 | 7/16 | 1/2 | 9/16 | 5/8 | 11/16 | 3/4 | 13/16 | 7/8 | 15/16 | 1 | 1 1/16 | 1 1/8 | Size |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 8″×8″ | .... | .... | .... | .... | .... | .... | 7.75 | 8.68 | 9.61 | 10.53 | 11.44 | 12.34 | 13.23 | 14.12 | 15.00 | 15.87 | 16.73 | 8″×8″ |
| 6 ×6 | .... | .... | .... | .... | 4.36 | 5.06 | 5.75 | 6.43 | 7.11 | 7.78 | 8.44 | 9.09 | 9.73 | 10.37 | 11.00 | .... | .... | 6 ×6 |
| 5 ×5 | .... | .... | .... | .... | 3.61 | 4.18 | 4.75 | 5.31 | 5.86 | 6.40 | 6.94 | 7.47 | 7.98 | 8.50 | 9.00 | .... | .... | 5 ×5 |
| 4 ×4 | .... | .... | .... | 2.40 | 2.86 | 3.31 | 3.75 | 4.18 | 4.61 | 5.03 | 5.44 | 5.84 | .... | .... | .... | .... | .... | 4 ×4 |
| 3½×3½ | .... | .... | .... | 2.09 | 2.48 | 2.87 | 3.25 | 3.62 | 3.98 | 4.34 | 4.69 | 5.03 | .... | .... | .... | .... | .... | 3½×3½ |
| 3 ×3 | .... | .... | 1.44 | 1.78 | 2.11 | 2.43 | 2.75 | 3.06 | 3.36 | .... | .... | .... | .... | .... | .... | .... | .... | 3 ×3 |
| 2¾×2¾ | .... | .... | 1.31 | 1.62 | 1.92 | 2.22 | 2.50 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 2¾×2¾ |
| 2½×2½ | .... | 0.90 | 1.19 | 1.47 | 1.73 | 2.00 | 2.25 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 2½×2½ |
| 2¼×2¼ | .... | 0.81 | 1.06 | 1.31 | 1.55 | 1.78 | 2.00 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 2¼×2¼ |
| 2 ×2 | .... | 0.71 | 0.94 | 1.15 | 1.36 | 1.56 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 2 ×2 |
| 1¾×1¾ | .... | 0.62 | 0.81 | 1.00 | 1.17 | 1.34 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1¾×1¾ |
| 1½×1½ | 0.36 | 0.53 | 0.69 | 0.84 | 0.98 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1½×1½ |
| 1¼×1¼ | 0.30 | 0.43 | 0.56 | 0.68 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1¼×1¼ |
| 1 ×1 | 0.23 | 0.34 | 0.44 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1 ×1 |

**Angles with Unequal Legs**

| Size | 1/8 | 3/16 | 1/4 | 5/16 | 3/8 | 7/16 | 1/2 | 9/16 | 5/8 | 11/16 | 3/4 | 13/16 | 7/8 | 15/16 | 1 | 1 1/16 | 1 1/8 | Size |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 7″×3½″ | .... | .... | .... | .... | .... | 4.40 | 5.00 | 5.59 | 6.17 | 6.75 | 7.31 | 7.87 | 8.42 | 8.97 | 9.50 | .... | .... | 7″×3½″ |
| 6 ×4 | .... | .... | .... | .... | 3.61 | 4.18 | 4.75 | 5.31 | 5.86 | 6.40 | 6.94 | 7.47 | 7.98 | 8.50 | 9.00 | .... | .... | 6 ×4 |
| 6 ×3½ | .... | .... | .... | .... | 3.42 | 3.97 | 4.50 | 5.03 | 5.55 | 6.06 | 6.56 | 7.06 | 7.55 | 8.03 | 8.50 | .... | .... | 6 ×3½ |
| 5 ×4 | .... | .... | .... | .... | 3.23 | 3.75 | 4.25 | 4.75 | 5.23 | 5.72 | 6.19 | 6.65 | 7.11 | .... | .... | .... | .... | 5 ×4 |
| 5 ×3½ | .... | .... | .... | 2.56 | 3.05 | 3.53 | 4.00 | 4.47 | 4.92 | 5.37 | 5.81 | 6.25 | 6.67 | .... | .... | .... | .... | 5 ×3½ |
| 5 ×3 | .... | .... | .... | 2.40 | 2.86 | 3.31 | 3.75 | 4.18 | 4.61 | 5.03 | 5.44 | 5.84 | .... | .... | .... | .... | .... | 5 ×3 |
| 4 ×3½ | .... | .... | .... | 2.25 | 2.67 | 3.09 | 3.50 | 3.90 | 4.30 | 4.68 | 5.06 | 5.43 | .... | .... | .... | .... | .... | 4 ×3½ |
| 4 ×3 | .... | .... | .... | 2.09 | 2.48 | 2.87 | 3.25 | 3.62 | 3.98 | 4.34 | 4.69 | 5.03 | .... | .... | .... | .... | .... | 4 ×3 |
| 3½×3 | .... | .... | .... | 1.93 | 2.30 | 2.65 | 3.00 | 3.34 | 3.67 | 4.00 | 4.31 | 4.62 | .... | .... | .... | .... | .... | 3½×3 |
| 3½×2½ | .... | .... | 1.44 | 1.78 | 2.11 | 2.43 | 2.75 | 3.06 | 3.36 | 3.65 | .... | .... | .... | .... | .... | .... | .... | 3½×2½ |
| 3 ×2½ | .... | .... | 1.31 | 1.62 | 1.92 | 2.22 | 2.50 | 2.78 | .... | .... | .... | .... | .... | .... | .... | .... | .... | 3 ×2½ |
| 3 ×2 | .... | .... | 1.19 | 1.47 | 1.73 | 2.00 | 2.25 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 3 ×2 |
| 2½×2 | .... | 0.81 | 1.06 | 1.31 | 1.55 | 1.78 | 2.00 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 2½×2 |
| Size | 1/8 | 3/16 | 1/4 | 5/16 | 3/8 | 7/16 | 1/2 | 9/16 | 5/8 | 11/16 | 3/4 | 13/16 | 7/8 | 15/16 | 1 | 1 1/16 | 1 1/8 | Size |

## TABLE 2.

### Weights of Angles

| Angles with Equal Legs | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Weights in Pounds per Foot Dimensions in Inches | | | | | | | | | | | | | | | | | | |
| Size | 1/8 | 3/16 | 1/4 | 5/16 | 3/8 | 7/16 | 1/2 | 9/16 | 5/8 | 11/16 | 3/4 | 13/16 | 7/8 | 15/16 | 1 | 1 1/16 | 1 1/8 | Size |
| 8″×8″ | .... | .... | .... | .... | .... | .... | 26.4 | 29.6 | 32.7 | 35.8 | 38.9 | 42.0 | 45.0 | 48.1 | 51.0 | 54.0 | 56.9 | 8″×8″ |
| 6 ×6 | .... | .... | .... | .... | 14.9 | 17.2 | 19.6 | 21.9 | 24.2 | 26.5 | 28.7 | 31.0 | 33.1 | 35.3 | 37.4 | .... | .... | 6 ×6 |
| 5 ×5 | .... | .... | .... | .... | 12.3 | 14.3 | 16.2 | 18.1 | 20.0 | 21.8 | 23.6 | 25.4 | 27.2 | 28.9 | 30.6 | .... | .... | 5 ×5 |
| 4 ×4 | .... | .... | .... | 8.2 | 9.8 | 11.3 | 12.8 | 14.3 | 15.7 | 17.1 | 18.5 | 19.9 | .... | .... | .... | .... | .... | 4 ×4 |
| 3½×3½ | .... | .... | .... | 7.2 | 8.5 | 9.8 | 11.1 | 12.4 | 13.6 | 14.8 | 16.0 | 17.1 | .... | .... | .... | .... | .... | 3½×3½ |
| 3 ×3 | .... | .... | 4.9 | 6.1 | 7.2 | 8.3 | 9.4 | 10.4 | 11.5 | .... | .... | .... | .... | .... | .... | .... | .... | 3 ×3 |
| 2¾×2¾ | .... | .... | 4.5 | 5.6 | 6.6 | 7.6 | 8.5 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 2¾×2¾ |
| 2½×2½ | .... | 3.1 | 4.1 | 5.0 | 5.9 | 6.8 | 7.7 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 2½×2½ |
| 2¼×2¼ | .... | 2.8 | 3.6 | 4.5 | 5.3 | 6.1 | 6.8 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 2¼×2¼ |
| 2 ×2 | .... | 2.4 | 3.2 | 3.9 | 4.7 | 5.3 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 2 ×2 |
| 1¾×1¾ | .... | 2.1 | 2.8 | 3.4 | 4.0 | 4.6 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1¾×1¾ |
| 1½×1½ | 1.2 | 1.8 | 3.3 | 2.9 | 3.4 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1½×1½ |
| 1¼×1¼ | 1.0 | 1.5 | 1.9 | 2.3 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1¼×1¼ |
| 1×1 | 0.8 | 1.2 | 1.5 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1 ×1 |
| Angles with Unequal Legs | | | | | | | | | | | | | | | | | | |
| Size | 1/8 | 3/16 | 1/4 | 5/16 | 3/8 | 7/16 | 1/2 | 9/16 | 5/8 | 11/16 | 3/4 | 13/16 | 7/8 | 15/16 | 1 | 1 1/16 | 1 1/8 | Size |
| 7″×3½″ | .... | .... | .... | .... | .... | 15.0 | 17.0 | 19.1 | 21.0 | 23.0 | 24.9 | 26.8 | 28.7 | 30.5 | 32.3 | .... | .... | 7″×3½″ |
| 6 ×4 | .... | .... | .... | .... | 12.3 | 14.3 | 16.2 | 18.1 | 20.0 | 21.8 | 23.6 | 25.4 | 27.2 | 28.9 | 30.6 | .... | .... | 6 ×4 |
| 6 ×3½ | .... | .... | .... | .... | 11.7 | 13.5 | 15.3 | 17.1 | 18.9 | 20.6 | 22.4 | 24.0 | 25.7 | 27.3 | 28.9 | .... | .... | 6 ×3½ |
| 5 ×4 | .... | .... | .... | .... | 11.0 | 12.8 | 14.5 | 16.2 | 17.8 | 19.5 | 21.1 | 22.7 | 24.2 | .... | .... | .... | .... | 5 ×4 |
| 5 ×3½ | .... | .... | .... | 8.7 | 10.4 | 12.0 | 13.6 | 15.2 | 16.8 | 18.3 | 19.8 | 21.3 | 22.7 | .... | .... | .... | .... | 5 ×3½ |
| 5 ×3 | .... | .... | .... | 8.2 | 9.8 | 11.3 | 12.8 | 14.3 | 15.7 | 17.1 | 18.5 | 19.9 | .... | .... | .... | .... | .... | 5 ×3 |
| 4 ×3½ | .... | .... | .... | 7.7 | 9.1 | 10.6 | 11.9 | 13.3 | 14.7 | 16.0 | 17.3 | 18.5 | .... | .... | .... | .... | .... | 4 ×3½ |
| 4 ×3 | .... | .... | .... | 7.2 | 8.5 | 9.8 | 11.1 | 12.4 | 13.6 | 14.8 | 16.0 | 17.1 | .... | .... | .... | .... | .... | 4 ×3 |
| 3½×3 | .... | .... | .... | 6.6 | 7.9 | 9.1 | 10.2 | 11.4 | 12.5 | 13.6 | 14.7 | 15.8 | .... | .... | .... | .... | .... | 3½×3 |
| 3½×2½ | .... | .... | 4.9 | 6.1 | 7.2 | 8.3 | 9.4 | 10.4 | 11.5 | 12.5 | .... | .... | .... | .... | .... | .... | .... | 3½×2½ |
| 3 ×2½ | .... | .... | 4.5 | 5.6 | 6.6 | 7.6 | 8.5 | 9.5 | .... | .... | .... | .... | .... | .... | .... | .... | .... | 3 ×2½ |
| 3 ×2 | .... | .... | 4.1 | 5.0 | 5.9 | 6.8 | 7.7 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 3 ×2 |
| 2½×2 | .... | 2.8 | 3.7 | 4.5 | 5.3 | 6.1 | 6.8 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 2½×2 |
| Size | 1/8 | 3/16 | 1/4 | 5/16 | 3/8 | 7/16 | 1/2 | 9/16 | 5/8 | 11/16 | 3/4 | 13/16 | 7/8 | 15/16 | 1 | 1 1/16 | 1 1/8 | Size |

# TABLE 3.

## Carnegie Angles.

### Net Areas and Allowable Tension Values in Thousands of Pounds.

Maximum Fiber Stress, 16,000 Pounds per Square Inch.

| Size, Inches. | Thickness, Inches. | Weight per Foot, Pounds. | Area, Inches². | Net Areas and Stresses—Two Holes Deducted. | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | ⅞ Inch Rivets. | | ¾ Inch Rivets. | | ⅝ Inch Rivets. | |
| | | | | Area, Inches². | Stress. | Area, Inches². | Stress. | Area, Inches². | Stress. |
| 8 × 8 | 1 | 51.0 | 15.00 | 13.00 | 208.0 | 13.25 | 212.0 | ........ | ..... |
| 8 × 8 | 15/16 | 48.1 | 14.12 | 12.24 | 195.8 | 12.48 | 199.7 | ........ | ..... |
| 8 × 8 | 7/8 | 45.0 | 13.23 | 11.48 | 183.7 | 11.70 | 187.2 | ........ | ..... |
| 8 × 8 | 13/16 | 42.0 | 12.34 | 10.72 | 171.5 | 10.92 | 174.7 | ........ | ..... |
| 8 × 8 | 3/4 | 38.9 | 11.44 | 9.94 | 159.0 | 10.13 | 162.1 | ........ | ..... |
| 8 × 8 | 11/16 | 35.8 | 10.53 | 9.16 | 146.6 | 9.33 | 149.3 | ........ | ..... |
| 8 × 8 | 5/8 | 32.7 | 9.61 | 8.36 | 133.8 | 8.52 | 136.3 | 8.67 | 138.7 |
| 8 × 8 | 9/16 | 29.6 | 8.68 | 7.55 | 120.8 | 7.70 | 123.2 | 7.84 | 125.4 |
| 8 × 8 | 1/2 | 26.4 | 7.75 | 6.75 | 108.0 | 6.87 | 109.9 | 7.00 | 112.0 |
| 8 × 6 | 1 | 44.2 | 13.00 | 11.00 | 176.0 | 11.25 | 180.0 | ........ | ..... |
| 8 × 6 | 15/16 | 41.7 | 12.25 | 10.37 | 165.9 | 10.61 | 169.8 | ........ | ..... |
| 8 × 6 | 7/8 | 39.1 | 11.48 | 9.73 | 155.7 | 9.95 | 159.2 | ........ | ..... |
| 8 × 6 | 13/16 | 36.5 | 10.72 | 9.10 | 145.6 | 9.30 | 148.8 | ........ | ..... |
| 8 × 6 | 3/4 | 33.8 | 9.94 | 8.44 | 135.0 | 8.63 | 138.1 | ........ | ..... |
| 8 × 6 | 11/16 | 31.2 | 9.15 | 7.78 | 124.5 | 7.95 | 127.2 | ........ | ..... |
| 8 × 6 | 5/8 | 28.5 | 8.36 | 7.11 | 113.8 | 7.27 | 116.3 | 7.42 | 118.7 |
| 8 × 6 | 9/16 | 25.7 | 7.56 | 6.43 | 102.9 | 6.58 | 105.3 | 6.72 | 107.5 |
| 8 × 6 | 1/2 | 23.0 | 6.75 | 5.75 | 92.0 | 5.87 | 93.9 | 6.00 | 96.0 |
| 8 × 6 | 7/16 | 20.2 | 5.93 | 5.05 | 80.8 | 5.16 | 82.6 | 5.27 | 84.3 |
| 6 × 6 | 7/8 | 33.1 | 9.73 | 7.98 | 127.7 | 8.20 | 131.2 | ........ | ..... |
| 6 × 6 | 13/16 | 31.0 | 9.09 | 7.47 | 119.5 | 7.67 | 122.7 | ........ | ..... |
| 6 × 6 | 3/4 | 28.7 | 8.44 | 6.94 | 111.0 | 7.13 | 114.1 | ........ | ..... |
| 6 × 6 | 11/16 | 26.5 | 7.78 | 6.41 | 102.6 | 6.58 | 105.3 | ........ | ..... |
| 6 × 6 | 5/8 | 24.2 | 7.11 | 5.86 | 93.8 | 6.02 | 96.3 | 6.17 | 98.7 |
| 6 × 6 | 9/16 | 21.9 | 6.43 | 5.30 | 84.8 | 5.45 | 87.2 | 5.59 | 89.4 |
| 6 × 6 | 1/2 | 19.6 | 5.75 | 4.75 | 76.0 | 4.87 | 77.9 | 5.00 | 80.0 |
| 6 × 6 | 7/16 | 17.2 | 5.06 | 4.18 | 66.9 | 4.29 | 68.6 | 4.40 | 70.4 |
| 6 × 6 | 3/8 | 14.9 | 4.36 | 3.61 | 57.8 | 3.70 | 59.2 | 3.80 | 60.8 |
| 6 × 4 | 7/8 | 27.2 | 7.98 | 6.23 | 99.7 | 6.45 | 103.2 | ........ | ..... |
| 6 × 4 | 13/16 | 25.4 | 7.47 | 5.85 | 93.6 | 6.05 | 96.8 | ........ | ..... |
| 6 × 4 | 3/4 | 23.6 | 6.94 | 5.44 | 87.0 | 5.63 | 90.1 | ........ | ..... |
| 6 × 4 | 11/16 | 21.8 | 6.40 | 5.03 | 80.5 | 5.20 | 83.2 | ........ | ..... |
| 6 × 4 | 5/8 | 20.0 | 5.86 | 4.61 | 73.8 | 4.77 | 76.3 | 4.92 | 78.7 |
| 6 × 4 | 9/16 | 18.1 | 5.31 | 4.18 | 66.9 | 4.33 | 69.3 | 4.47 | 71.5 |
| 6 × 4 | 1/2 | 16.2 | 4.75 | 3.75 | 60.0 | 3.87 | 61.9 | 4.00 | 64.0 |
| 6 × 4 | 7/16 | 14.3 | 4.18 | 3.30 | 52.8 | 3.41 | 54.6 | 3.52 | 56.3 |
| 6 × 4 | 3/8 | 12.3 | 3.61 | 2.86 | 45.8 | 2.95 | 47.2 | 3.05 | 48.8 |
| 5 × 3½ | 5/8 | 16.8 | 4.92 | 3.67 | 58.7 | 3.83 | 61.3 | 3.98 | 63.7 |
| 5 × 3½ | 9/16 | 15.2 | 4.47 | 3.34 | 53.4 | 3.49 | 55.8 | 3.63 | 58.1 |
| 5 × 3½ | 1/2 | 13.6 | 4.00 | 3.00 | 48.0 | 3.12 | 49.9 | 3.25 | 52.0 |
| 5 × 3½ | 7/16 | 12.0 | 3.53 | 2.65 | 42.4 | 2.76 | 44.2 | 2.87 | 45.9 |
| 5 × 3½ | 3/8 | 10.4 | 3.05 | 2.30 | 36.8 | 2.39 | 38.2 | 2.49 | 39.8 |
| 5 × 3½ | 5/16 | 8.7 | 2.56 | 1.93 | 30.9 | 2.01 | 32.2 | 2.09 | 33.4 |
| 5 × 3 | 1/2 | 12.8 | 3.75 | 2.75 | 44.0 | 2.87 | 45.9 | 3.00 | 48.0 |
| 5 × 3 | 7/16 | 11.3 | 3.31 | 2.43 | 38.9 | 2.54 | 40.6 | 2.65 | 42.4 |
| 5 × 3 | 3/8 | 9.8 | 2.86 | 2.11 | 33.8 | 2.20 | 35.2 | 2.30 | 36.8 |
| 5 × 3 | 5/16 | 8.2 | 2.40 | 1.77 | 28.3 | 1.85 | 29.6 | 1.93 | 30.9 |

TABLE 3.—*Continued.*

CARNEGIE ANGLES.

NET AREAS AND ALLOWABLE TENSION VALUES IN THOUSANDS OF POUNDS.
Maximum Fiber Stress, 16,000 Pounds per Square Inch.

| Size, Inches. | Thickness, Inches. | Weight per Foot, Pounds. | Area, Inches². | Net Areas and Stresses—One Hole Deducted. | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | ⅞ Inch Rivets. | | ¾ Inch Rivets. | | ½ Inch Rivets. | |
| | | | | Area, Inches². | Stress. | Area, Inches². | Stress. | Area, Inches². | Stress. |
| 6 × 6 | ⅞ | 33.1 | 9.73 | 8.85 | 141.6 | 8.96 | 143.4 | ........ | ...... |
| 6 × 6 | 13/16 | 31.0 | 9.09 | 8.28 | 132.5 | 8.38 | 134.1 | ........ | ...... |
| 6 × 6 | ¾ | 28.7 | 8.44 | 7.69 | 123.0 | 7.78 | 124.5 | ........ | ...... |
| 6 × 6 | 11/16 | 26.5 | 7.78 | 7.09 | 113.4 | 7.18 | 114.9 | ........ | ...... |
| 6 × 6 | ⅝ | 24.2 | 7.11 | 6.48 | 103.7 | 6.56 | 105.0 | 6.64 | 106.2 |
| 6 × 6 | 9/16 | 21.9 | 6.43 | 5.87 | 93.9 | 5.94 | 95.0 | 6.01 | 96.2 |
| 6 × 6 | ½ | 19.6 | 5.75 | 5.25 | 84.0 | 5.31 | 85.0 | 5.37 | 85.9 |
| 6 × 6 | 7/16 | 17.2 | 5.06 | 4.62 | 73.9 | 4.68 | 74.9 | 4.73 | 75.7 |
| 6 × 6 | ⅜ | 14.9 | 4.36 | 3.98 | 63.7 | 4.03 | 64.5 | 4.08 | 65.3 |
| 6 × 4 | ⅞ | 27.2 | 7.98 | 7.10 | 113.6 | 7.21 | 115.4 | ........ | ...... |
| 6 × 4 | 13/16 | 25.4 | 7.47 | 6.66 | 106.6 | 6.76 | 108.2 | ........ | ...... |
| 6 × 4 | ¾ | 23.6 | 6.94 | 6.19 | 99.0 | 6.28 | 100.5 | ........ | ...... |
| 6 × 4 | 11/16 | 21.8 | 6.40 | 5.71 | 91.4 | 5.80 | 92.8 | ........ | ...... |
| 6 × 4 | ⅝ | 20.0 | 5.86 | 5.23 | 83.7 | 5.31 | 85.0 | 5.39 | 86.2 |
| 6 × 4 | 9/16 | 18.1 | 5.31 | 4.75 | 76.0 | 4.82 | 77.1 | 4.89 | 78.2 |
| 6 × 4 | ½ | 16.2 | 4.75 | 4.25 | 68.0 | 4.31 | 69.0 | 4.37 | 69.9 |
| 6 × 4 | 7/16 | 14.3 | 4.18 | 3.74 | 59.8 | 3.80 | 60.8 | 3.85 | 61.6 |
| 6 × 4 | ⅜ | 12.3 | 3.61 | 3.23 | 51.7 | 3.28 | 52.5 | 3.33 | 53.3 |
| 5 × 3½ | ⅝ | 16.8 | 4.92 | 4.29 | 68.6 | 4.37 | 69.9 | 4.45 | 71.2 |
| 5 × 3½ | 9/16 | 15.2 | 4.47 | 3.91 | 62.6 | 3.98 | 63.7 | 4.05 | 64.8 |
| 5 × 3½ | ½ | 13.6 | 4.00 | 3.50 | 56.0 | 3.56 | 57.0 | 3.62 | 57.9 |
| 5 × 3½ | 7/16 | 12.0 | 3.53 | 3.09 | 49.4 | 3.15 | 50.4 | 3.20 | 51.2 |
| 5 × 3½ | ⅜ | 10.4 | 3.05 | 2.67 | 42.7 | 2.72 | 43.5 | 2.77 | 44.3 |
| 5 × 3½ | 5/16 | 8.7 | 2.56 | 2.25 | 36.0 | 2.29 | 36.6 | 2.33 | 37.3 |
| 5 × 3 | ⅝ | 15.7 | 4.61 | 3.98 | 63.7 | 4.06 | 65.0 | 4.14 | 66.2 |
| 5 × 3 | 9/16 | 14.3 | 4.18 | 3.62 | 57.9 | 3.69 | 59.0 | 3.76 | 60.2 |
| 5 × 3 | ½ | 12.8 | 3.75 | 3.25 | 52.0 | 3.31 | 53.0 | 3.37 | 53.9 |
| 5 × 3 | 7/16 | 11.3 | 3.31 | 2.87 | 45.9 | 2.93 | 46.9 | 2.98 | 47.7 |
| 5 × 3 | ⅜ | 9.8 | 2.86 | 2.48 | 39.7 | 2.53 | 40.5 | 2.58 | 41.3 |
| 5 × 3 | 5/16 | 8.2 | 2.40 | 2.09 | 33.4 | 2.13 | 34.1 | 2.17 | 34.7 |
| 4 × 4 | ⅝ | 15.7 | 4.61 | 3.98 | 63.7 | 4.06 | 65.0 | 4.14 | 66.2 |
| 4 × 4 | 9/16 | 14.3 | 4.18 | 3.62 | 57.9 | 3.69 | 59.0 | 3.76 | 60.2 |
| 4 × 4 | ½ | 12.8 | 3.75 | 3.25 | 52.0 | 3.31 | 53.0 | 3.37 | 53.9 |
| 4 × 4 | 7/16 | 11.3 | 3.31 | 2.87 | 45.9 | 2.93 | 46.9 | 2.98 | 47.7 |
| 4 × 4 | ⅜ | 9.8 | 2.86 | 2.48 | 39.7 | 2.53 | 40.5 | 2.58 | 41.3 |
| 4 × 4 | 5/16 | 8.2 | 2.40 | 2.09 | 33.4 | 2.13 | 34.1 | 2.17 | 34.7 |
| 4 × 4 | ¼ | 6.6 | 1.94 | 1.69 | 27.0 | 1.72 | 27.5 | 1.75 | 28.0 |
| 4 × 3 | ½ | 11.1 | 3.25 | 2.75 | 44.0 | 2.81 | 45.0 | 2.87 | 45.9 |
| 4 × 3 | 7/16 | 9.8 | 2.87 | 2.43 | 38.9 | 2.49 | 39.8 | 2.54 | 40.6 |
| 4 × 3 | ⅜ | 8.5 | 2.48 | 2.10 | 33.6 | 2.15 | 34.4 | 2.20 | 35.2 |
| 4 × 3 | 5/16 | 7.2 | 2.09 | 1.78 | 28.5 | 1.82 | 29.1 | 1.86 | 29.8 |
| 4 × 3 | ¼ | 5.8 | 1.69 | 1.44 | 23.0 | 1.47 | 23.5 | 1.50 | 24.0 |

TABLE 3.—*Continued.*

CARNEGIE ANGLES.

NET AREAS AND ALLOWABLE TENSION VALUES IN THOUSANDS OF POUNDS.

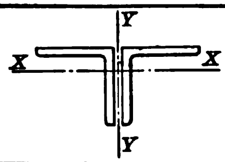
Maximum Fiber Stress, 16,000 Pounds per Square Inch.

| Size, Inches. | Thickness, Inches. | Weight per Foot, Pounds. | Area, Inches². | Net Areas and Stresses—One Hole Deducted. | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | ⅞ Inch Rivets. | | ¾ Inch Rivets. | | ⅝ Inch Rivets. | |
| | | | | Area, Inches². | Stress. | Area, Inches². | Stress. | Area, Inches². | Stress. |
| 3½ × 3½ | ⅝ | 13.6 | 3.98 | 3.35 | 53.6 | 3.43 | 54.9 | 3.51 | 56.2 |
| 3½ × 3½ | 9/16 | 12.4 | 3.62 | 3.06 | 49.0 | 3.13 | 50.1 | 3.20 | 51.2 |
| 3½ × 3½ | ½ | 11.1 | 3.25 | 2.75 | 44.0 | 2.81 | 45.0 | 2.87 | 45.9 |
| 3½ × 3½ | 7/16 | 9.8 | 2.87 | 2.43 | 38.9 | 2.49 | 39.8 | 2.54 | 40.6 |
| 3½ × 3½ | ⅜ | 8.5 | 2.48 | 2.10 | 33.6 | 2.15 | 34.4 | 2.20 | 35.2 |
| 3½ × 3½ | 5/16 | 7.2 | 2.09 | 1.78 | 28.5 | 1.82 | 29.1 | 1.86 | 29.8 |
| 3½ × 3½ | ¼ | 5.8 | 1.69 | 1.44 | 23.0 | 1.47 | 23.5 | 1.50 | 24.0 |
| 3½ × 3 | ½ | 10.2 | 3.00 | 2.50 | 40.0 | 2.56 | 41.0 | 2.62 | 41.9 |
| 3½ × 3 | 7/16 | 9.1 | 2.65 | 2.21 | 35.4 | 2.27 | 36.3 | 2.32 | 37.1 |
| 3½ × 3 | ⅜ | 7.9 | 2.30 | 1.92 | 30.7 | 1.97 | 31.5 | 2.02 | 32.3 |
| 3½ × 3 | 5/16 | 6.6 | 1.93 | 1.62 | 25.9 | 1.66 | 26.6 | 1.70 | 27.2 |
| 3½ × 3 | ¼ | 5.4 | 1.56 | 1.31 | 21.0 | 1.34 | 21.4 | 1.37 | 21.9 |
| 3½ × 2½ | ½ | 9.4 | 2.75 | 2.25 | 36.0 | 2.31 | 37.0 | 2.37 | 37.9 |
| 3½ × 2½ | 7/16 | 8.3 | 2.43 | 1.99 | 31.8 | 2.05 | 32.8 | 2.10 | 33.6 |
| 3½ × 2½ | ⅜ | 7.2 | 2.11 | 1.73 | 27.7 | 1.78 | 28.5 | 1.83 | 29.3 |
| 3½ × 2½ | 5/16 | 6.1 | 1.78 | 1.47 | 23.5 | 1.51 | 24.2 | 1.55 | 24.8 |
| 3½ × 2½ | ¼ | 4.9 | 1.44 | 1.19 | 19.0 | 1.22 | 19.5 | 1.25 | 20.0 |
| 3 × 3 | ½ | 9.4 | 2.75 | 2.25 | 36.0 | 2.31 | 37.0 | 2.37 | 37.9 |
| 3 × 3 | 7/16 | 8.3 | 2.43 | 1.99 | 31.8 | 2.05 | 32.8 | 2.10 | 33.6 |
| 3 × 3 | ⅜ | 7.2 | 2.11 | 1.73 | 27.7 | 1.78 | 28.5 | 1.83 | 29.3 |
| 3 × 3 | 5/16 | 6.1 | 1.78 | 1.47 | 23.5 | 1.51 | 24.2 | 1.55 | 24.8 |
| 3 × 3 | ¼ | 4.9 | 1.44 | 1.19 | 19.0 | 1.22 | 19.5 | 1.25 | 20.0 |
| 3 × 2½ | ⅜ | 6.6 | 1.92 | 1.54 | 24.6 | 1.59 | 25.4 | 1.64 | 26.2 |
| 3 × 2½ | 5/16 | 5.6 | 1.62 | 1.31 | 21.0 | 1.35 | 21.6 | 1.39 | 22.2 |
| 3 × 2½ | ¼ | 4.5 | 1.31 | 1.06 | 17.0 | 1.09 | 17.4 | 1.12 | 17.9 |
| 2½ × 2½ | ⅜ | 5.9 | 1.73 | ........ | ........ | 1.40 | 22.4 | 1.45 | 23.2 |
| 2½ × 2½ | 5/16 | 5.0 | 1.47 | ........ | ........ | 1.20 | 19.2 | 1.24 | 19.8 |
| 2½ × 2½ | ¼ | 4.1 | 1.19 | ........ | ........ | 0.97 | 15.5 | 1.00 | 16.0 |
| 2½ × 2½ | 3/16 | 3.07 | 0.90 | ........ | ........ | 0.74 | 11.8 | 0.76 | 12.2 |
| 2½ × 2 | ⅜ | 5.3 | 1.55 | ........ | ........ | 1.22 | 19.5 | 1.27 | 20.3 |
| 2½ × 2 | 5/16 | 4.5 | 1.31 | ........ | ........ | 1.04 | 16.6 | 1.08 | 17.3 |
| 2½ × 2 | ¼ | 3.62 | 1.06 | ........ | ........ | 0.84 | 13.4 | 0.87 | 13.9 |
| 2½ × 2 | 3/16 | 2.75 | 0.81 | ........ | ........ | 0.65 | 10.4 | 0.67 | 10.7 |
| 2 × 2 | ⅜ | 4.7 | 1.36 | ........ | ........ | ........ | ........ | 1.08 | 17.3 |
| 2 × 2 | 5/16 | 3.92 | 1.15 | ........ | ........ | ........ | ........ | 0.92 | 14.7 |
| 2 × 2 | ¼ | 3.19 | 0.94 | ........ | ........ | ........ | ........ | 0.75 | 12.0 |
| 2 × 2 | 3/16 | 2.44 | 0.71 | ........ | ........ | ........ | ........ | 0.57 | 9.1 |
| 2 × 1½ | 5/16 | 3.39 | 1.00 | ........ | ........ | ........ | ........ | 0.77 | 12.3 |
| 2 × 1½ | ¼ | 2.77 | 0.81 | ........ | ........ | ........ | ........ | 0.62 | 9.9 |
| 2 × 1½ | 3/16 | 2.12 | 0.62 | ........ | ........ | ........ | ........ | 0.48 | 7.7 |

## TABLE 4.

### RADII OF GYRATION OF TWO ANGLES WITH EQUAL LEGS, BOTH AXES.

**Radii of Gyration of Two Angles, Equal Legs.**




**For Distances Measured from Back to Back.**

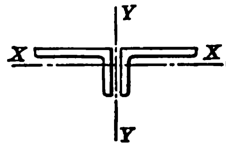
| Size of Angles. | Area, Two Angles. | Axis X-X. | Axis Y-Y. Distance Back to Back in Inches. | | | | | | | Size of Angles. | Area, Two Angles. | Axis X-X. | Axis Y-Y. Distance Back to Back in Inches. | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| In. | In.² | | 0 | 1/4 | 5/16 | 3/8 | 1/2 | 5/8 | 3/4 | In. | In.² | | 0 | 1/4 | 5/16 | 3/8 | 1/2 | 5/8 | 3/4 |
| 2x2x3/16 | 1.42 | .62 | .84 | .93 | .95 | .99 | 1.04 | 1.09 | 1.14 | 2 1/2x2 1/2x1/4 | 2.38 | .77 | 1.05 | 1.14 | 1.17 | 1.19 | 1.24 | 1.29 | 1.34 |
| " 1/4 | 1.88 | .61 | .85 | .94 | .96 | .99 | 1.04 | 1.09 | 1.14 | " 5/16 | 2.94 | .76 | 1.06 | 1.15 | 1.17 | 1.20 | 1.25 | 1.30 | 1.35 |
| " 5/16 | 2.30 | .60 | .86 | .95 | .98 | 1.00 | 1.05 | 1.10 | 1.15 | " 3/8 | 3.46 | .75 | 1.07 | 1.16 | 1.18 | 1.21 | 1.26 | 1.31 | 1.36 |
| " 3/8 | 2.72 | .59 | .88 | .97 | .99 | 1.01 | 1.07 | 1.11 | 1.16 | " 7/16 | 4.00 | .75 | 1.08 | 1.17 | 1.20 | 1.22 | 1.27 | 1.32 | 1.37 |
| 3x3x1/4 | 2.88 | .93 | 1.25 | 1.34 | 1.36 | 1.38 | 1.43 | 1.48 | 1.53 | 3 1/2x3 1/2x1/4 | 3.38 | 1.09 | 1.45 | 1.54 | 1.57 | 1.59 | 1.63 | 1.67 | 1.73 |
| " 5/16 | 3.56 | .92 | 1.26 | 1.36 | 1.38 | 1.40 | 1.45 | 1.50 | 1.55 | " 5/16 | 4.18 | 1.08 | 1.47 | 1.56 | 1.58 | 1.60 | 1.65 | 1.69 | 1.74 |
| " 3/8 | 4.22 | .91 | 1.27 | 1.37 | 1.39 | 1.41 | 1.46 | 1.51 | 1.56 | " 3/8 | 4.96 | 1.07 | 1.48 | 1.57 | 1.59 | 1.61 | 1.66 | 1.70 | 1.75 |
| " 7/16 | 4.86 | .91 | 1.28 | 1.38 | 1.40 | 1.42 | 1.47 | 1.52 | 1.57 | " 7/16 | 5.74 | 1.07 | 1.49 | 1.58 | 1.60 | 1.62 | 1.67 | 1.72 | 1.77 |
| " 1/2 | 5.50 | .90 | 1.29 | 1.39 | 1.41 | 1.43 | 1.48 | 1.53 | 1.58 | " 1/2 | 6.50 | 1.06 | 1.50 | 1.59 | 1.61 | 1.63 | 1.67 | 1.73 | 1.78 |
| " 9/16 | 6.12 | .89 | 1.30 | 1.40 | 1.42 | 1.45 | 1.50 | 1.54 | 1.60 | " 9/16 | 7.24 | 1.05 | 1.51 | 1.60 | 1.62 | 1.64 | 1.69 | 1.75 | 1.80 |
| " 5/8 | 6.72 | .88 | 1.32 | 1.41 | 1.43 | 1.46 | 1.51 | 1.55 | 1.62 | " 5/8 | 7.96 | 1.04 | 1.52 | 1.61 | 1.63 | 1.66 | 1.70 | 1.76 | 1.81 |

| Size of Angles. | Area, Two Angles. | Axis X-X. | Axis Y-Y. Distance Back to Back of Angles in Inches. | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| In. | In.² | | 0 | 1/4 | 5/16 | 3/8 | 7/16 | 1/2 | 9/16 | 5/8 | 3/4 | 7/8 | 1 | 1 1/8 | 1 1/4 |
| 4x4x1/4 | 3.88 | 1.25 | 1.66 | 1.75 | 1.77 | 1.79 | 1.82 | 1.84 | 1.86 | 1.88 | 1.93 | ...... | ...... | ...... | ...... |
| " 5/16 | 4.80 | 1.24 | 1.68 | 1.76 | 1.78 | 1.80 | 1.83 | 1.85 | 1.87 | 1.89 | 1.94 | ...... | ...... | ...... | ...... |
| " 3/8 | 5.72 | 1.23 | 1.68 | 1.77 | 1.79 | 1.81 | 1.84 | 1.86 | 1.88 | 1.90 | 1.95 | ...... | ...... | ...... | ...... |
| " 7/16 | 6.62 | 1.23 | 1.69 | 1.78 | 1.80 | 1.82 | 1.85 | 1.87 | 1.89 | 1.92 | 1.96 | ...... | ...... | ...... | ...... |
| " 1/2 | 7.50 | 1.22 | 1.70 | 1.79 | 1.81 | 1.83 | 1.86 | 1.88 | 1.90 | 1.93 | 1.97 | ...... | ...... | ...... | ...... |
| " 9/16 | 8.36 | 1.21 | 1.71 | 1.80 | 1.82 | 1.85 | 1.87 | 1.90 | 1.92 | 1.94 | 1.99 | ...... | ...... | ...... | ...... |
| " 5/8 | 9.22 | 1.20 | 1.72 | 1.81 | 1.83 | 1.86 | 1.88 | 1.91 | 1.93 | 1.95 | 2.00 | ...... | ...... | ...... | ...... |
| 5x5x3/8 | 7.22 | 1.56 | 2.08 | 2.17 | 2.19 | 2.22 | 2.24 | 2.26 | 2.28 | 2.31 | 2.35 | ...... | ...... | ...... | ...... |
| " 7/16 | 8.36 | 1.55 | 2.09 | 2.18 | 2.20 | 2.22 | 2.25 | 2.27 | 2.29 | 2.32 | 2.37 | ...... | ...... | ...... | ...... |
| " 1/2 | 9.50 | 1.54 | 2.10 | 2.19 | 2.21 | 2.23 | 2.26 | 2.28 | 2.30 | 2.33 | 2.38 | ...... | ...... | ...... | ...... |
| " 9/16 | 10.62 | 1.53 | 2.11 | 2.20 | 2.22 | 2.25 | 2.27 | 2.29 | 2.32 | 2.34 | 2.39 | ...... | ...... | ...... | ...... |
| " 5/8 | 11.72 | 1.52 | 2.12 | 2.21 | 2.23 | 2.26 | 2.28 | 2.30 | 2.33 | 2.35 | 2.40 | ...... | ...... | ...... | ...... |
| " 11/16 | 12.80 | 1.51 | 2.13 | 2.22 | 2.24 | 2.27 | 2.29 | 2.32 | 2.34 | 2.36 | 2.41 | ...... | ...... | ...... | ...... |
| " 3/4 | 13.88 | 1.50 | 2.14 | 2.23 | 2.25 | 2.28 | 2.30 | 2.33 | 2.35 | 2.37 | 2.42 | ...... | ...... | ...... | ...... |
| 6x6x3/8 | 8.72 | 1.88 | 2.49 | ...... | ...... | 2.62 | 2.64 | 2.66 | 2.69 | 2.71 | 2.75 | 2.80 | 2.85 | 2.90 | 2.94 |
| " 7/16 | 10.12 | 1.87 | 2.50 | ...... | ...... | 2.63 | 2.65 | 2.67 | 2.69 | 2.72 | 2.76 | 2.81 | 2.86 | 2.91 | 2.95 |
| " 1/2 | 11.50 | 1.86 | 2.51 | ...... | ...... | 2.64 | 2.66 | 2.68 | 2.71 | 2.73 | 2.77 | 2.82 | 2.87 | 2.91 | 2.96 |
| " 9/16 | 12.86 | 1.85 | 2.52 | ...... | ...... | 2.65 | 2.67 | 2.70 | 2.72 | 2.74 | 2.79 | 2.84 | 2.88 | 2.93 | 2.98 |
| " 5/8 | 14.22 | 1.84 | 2.53 | ...... | ...... | 2.66 | 2.68 | 2.71 | 2.73 | 2.75 | 2.80 | 2.85 | 2.89 | 2.94 | 2.99 |
| " 11/16 | 15.56 | 1.83 | 2.53 | ...... | ...... | 2.67 | 2.69 | 2.71 | 2.74 | 2.76 | 2.81 | 2.85 | 2.90 | 2.95 | 3.00 |
| " 3/4 | 16.88 | 1.83 | 2.55 | ...... | ...... | 2.68 | 2.71 | 2.73 | 2.76 | 2.78 | 2.83 | 2.88 | 2.92 | 2.97 | 3.02 |
| " 7/8 | 19.46 | 1.81 | 2.57 | ...... | ...... | 2.70 | 2.73 | 2.75 | 2.77 | 2.80 | 2.85 | 2.90 | 2.94 | 2.99 | 3.04 |
| " 1 | 22.00 | 1.80 | 2.59 | ...... | ...... | 2.72 | 2.75 | 2.77 | 2.79 | 2.82 | 2.87 | 2.92 | 2.97 | 3.01 | 3.06 |
| 8x8x1/2 | 15.50 | 2.51 | 3.32 | ...... | ...... | 3.44 | 3.47 | 3.49 | 3.52 | 3.54 | 3.58 | 3.63 | 3.67 | 3.72 | 3.77 |
| " 9/16 | 17.36 | 2.50 | 3.33 | ...... | ...... | 3.46 | 3.48 | 3.50 | 3.53 | 3.55 | 3.59 | 3.64 | 3.68 | 3.73 | 3.78 |
| " 5/8 | 19.22 | 2.49 | 3.34 | ...... | ...... | 3.47 | 3.49 | 3.51 | 3.53 | 3.56 | 3.60 | 3.64 | 3.69 | 3.74 | 3.78 |
| " 11/16 | 21.06 | 2.48 | 3.35 | ...... | ...... | 3.48 | 3.50 | 3.52 | 3.54 | 3.57 | 3.61 | 3.65 | 3.70 | 3.75 | 3.79 |
| " 3/4 | 22.88 | 2.47 | 3.36 | ...... | ...... | 3.49 | 3.51 | 3.53 | 3.56 | 3.58 | 3.62 | 3.67 | 3.72 | 3.76 | 3.81 |
| " 7/8 | 26.46 | 2.45 | 3.38 | ...... | ...... | 3.51 | 3.53 | 3.55 | 3.57 | 3.60 | 3.64 | 3.69 | 3.74 | 3.78 | 3.83 |
| " 1 | 30.00 | 2.44 | 3.40 | ...... | ...... | 3.53 | 3.55 | 3.57 | 3.60 | 3.62 | 3.67 | 3.71 | 3.76 | 3.81 | 3.86 |
| " 1 1/8 | 33.46 | 2.42 | 3.42 | ...... | ...... | 3.55 | 3.57 | 3.60 | 3.62 | 3.64 | 3.69 | 3.74 | 3.79 | 3.83 | .388 |

## TABLE 5.

### Radii of Gyration of Two Angles with Unequal Legs, Both Axes. Long Legs Out.

Radii of Gyration of Two Angles, Long Legs Turned Out.




For Distances Measured from Back to Back.

| Size of Angles. | Area, Two Angles. | Axis X-X. | Axis Y-Y. Distance Back to Back in Inches. | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| In. | In.² | | 0 | ¼ | 5/16 | ⅜ | ½ | ⅝ | ¾ |
| 2½x2x3/16 | 1.62 | .60 | 1.10 | 1.19 | 1.22 | 1.24 | 1.29 | 1.34 | 1.38 |
| " ¼ | 2.12 | .59 | 1.11 | 1.20 | 1.23 | 1.25 | 1.30 | 1.36 | 1.40 |
| " 5/16 | 2.62 | .58 | 1.12 | 1.21 | 1.24 | 1.26 | 1.31 | 1.37 | 1.42 |
| " ⅜ | 3.10 | .58 | 1.13 | 1.22 | 1.25 | 1.28 | 1.32 | 1.38 | 1.44 |
| " 7/16 | 3.56 | .57 | 1.14 | 1.24 | 1.26 | 1.29 | 1.33 | 1.39 | 1.46 |
| 3½x2½x¼ | 2.88 | .74 | 1.58 | 1.67 | 1.69 | 1.71 | 1.76 | 1.81 | 1.86 |
| " 5/16 | 3.56 | .73 | 1.60 | 1.68 | 1.70 | 1.73 | 1.77 | 1.82 | 1.88 |
| " ⅜ | 4.22 | .72 | 1.61 | 1.69 | 1.72 | 1.74 | 1.79 | 1.84 | 1.89 |
| " 7/16 | 4.86 | .71 | 1.61 | 1.70 | 1.73 | 1.75 | 1.80 | 1.85 | 1.90 |
| " ½ | 5.50 | .70 | 1.62 | 1.71 | 1.74 | 1.76 | 1.81 | 1.86 | 1.91 |
| 4x3x¼ | 3.38 | .89 | 1.77 | 1.87 | 1.89 | 1.92 | 1.96 | 2.01 | 2.06 |
| " 5/16 | 4.18 | .89 | 1.79 | 1.88 | 1.90 | 1.93 | 1.97 | 2.02 | 2.07 |
| " ⅜ | 4.96 | .88 | 1.80 | 1.89 | 1.91 | 1.94 | 1.98 | 2.03 | 2.08 |
| " 7/16 | 5.74 | .87 | 1.81 | 1.90 | 1.92 | 1.95 | 1.99 | 2.04 | 2.09 |
| " ½ | 6.50 | .86 | 1.82 | 1.92 | 1.94 | 1.96 | 2.01 | 2.06 | 2.11 |
| " 9/16 | 7.24 | .86 | 1.83 | 1.93 | 1.95 | 1.97 | 2.02 | 2.07 | 2.12 |
| " ⅝ | 7.96 | .85 | 1.84 | 1.94 | 1.96 | 1.98 | 2.03 | 2.08 | 2.14 |

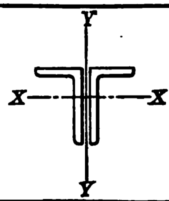
| Size of Angles. | Area, Two Angles. | Axis X-X. | Axis Y-Y. Distance Back to Back in Inches. | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| In. | In.² | | 0 | ¼ | 5/16 | ⅜ | ½ | ⅝ | ¾ |
| 3x2½x¼ | 2.62 | .75 | 1.31 | 1.40 | 1.42 | 1.45 | 1.50 | 1.55 | 1.59 |
| " 5/16 | 3.24 | .74 | 1.32 | 1.41 | 1.43 | 1.46 | 1.51 | 1.56 | 1.60 |
| " ⅜ | 3.84 | .74 | 1.33 | 1.43 | 1.45 | 1.48 | 1.53 | 1.58 | 1.62 |
| " 7/16 | 4.44 | .73 | 1.34 | 1.44 | 1.46 | 1.49 | 1.54 | 1.59 | 1.64 |
| " ½ | 5.00 | .72 | 1.35 | 1.45 | 1.47 | 1.50 | 1.55 | 1.60 | 1.65 |
| 3½x3x¼ | 3.12 | .91 | 1.52 | 1.61 | 1.63 | 1.65 | 1.70 | 1.75 | 1.79 |
| " 5/16 | 3.86 | .90 | 1.52 | 1.61 | 1.64 | 1.66 | 1.71 | 1.76 | 1.81 |
| " ⅜ | 4.60 | .90 | 1.53 | 1.62 | 1.65 | 1.67 | 1.72 | 1.77 | 1.82 |
| " 7/16 | 5.30 | .89 | 1.54 | 1.63 | 1.66 | 1.68 | 1.73 | 1.78 | 1.83 |
| " ½ | 6.00 | .88 | 1.55 | 1.65 | 1.68 | 1.70 | 1.75 | 1.80 | 1.85 |
| 5x3x5/16 | 4.80 | .85 | 2.33 | 2.42 | 2.45 | 2.47 | 2.52 | 2.57 | 2.62 |
| " ⅜ | 5.72 | .84 | 2.34 | 2.43 | 2.46 | 2.48 | 2.53 | 2.58 | 2.63 |
| " 7/16 | 6.62 | .84 | 2.35 | 2.45 | 2.47 | 2.49 | 2.54 | 2.59 | 2.64 |
| " ½ | 7.50 | .83 | 2.36 | 2.46 | 2.48 | 2.50 | 2.55 | 2.60 | 2.65 |
| " 9/16 | 8.36 | .82 | 2.37 | 2.47 | 2.49 | 2.52 | 2.57 | 2.61 | 2.66 |
| " ⅝ | 9.22 | .82 | 2.39 | 2.48 | 2.51 | 2.53 | 2.58 | 2.63 | 2.68 |
| " 11/16 | 10.06 | .81 | 2.40 | 2.49 | 2.52 | 2.54 | 2.59 | 2.64 | 2.69 |

| Size of Angles. | Area, Two Angles. | Axis X-X. | Axis Y-Y. Distance Back to Back of Angles in Inches. | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| In. | In.² | | 0 | ¼ | 5/16 | ⅜ | 7/16 | ½ | 9/16 | ⅝ | ¾ | ⅞ | 1 | 1⅛ | 1¼ |
| 5x3½x5/16 | 5.12 | 1.03 | 2.26 | 2.35 | 2.37 | 2.39 | 2.42 | 2.44 | 2.47 | 2.49 | 2.54 | ...... | ...... | ...... | ...... |
| " ⅜ | 6.10 | 1.02 | 2.27 | 2.36 | 2.38 | 2.40 | 2.43 | 2.45 | 2.48 | 2.50 | 2.55 | ...... | ...... | ...... | ...... |
| " 7/16 | 7.06 | 1.01 | 2.28 | 2.37 | 2.39 | 2.41 | 2.44 | 2.46 | 2.49 | 2.52 | 2.56 | ...... | ...... | ...... | ...... |
| " ½ | 8.00 | 1.01 | 2.29 | 2.38 | 2.41 | 2.43 | 2.45 | 2.48 | 2.50 | 2.53 | 2.58 | ...... | ...... | ...... | ...... |
| " 9/16 | 8.94 | 1.00 | 2.30 | 2.39 | 2.42 | 2.44 | 2.46 | 2.49 | 2.51 | 2.54 | 2.59 | ...... | ...... | ...... | ...... |
| " ⅝ | 9.84 | .99 | 2.31 | 2.40 | 2.43 | 2.45 | 2.48 | 2.50 | 2.52 | 2.55 | 2.60 | ...... | ...... | ...... | ...... |
| " 11/16 | 10.74 | .98 | 2.32 | 2.41 | 2.44 | 2.46 | 2.49 | 2.51 | 2.53 | 2.56 | 2.61 | ...... | ...... | ...... | ...... |
| " ¾ | 11.62 | .98 | 2.33 | 2.43 | 2.46 | 2.48 | 2.51 | 2.53 | 2.55 | 2.58 | 2.63 | ...... | ...... | ...... | ...... |
| 6x4x⅜ | 7.22 | 1.17 | 2.74 | 2.83 | 2.85 | 2.87 | 2.90 | 2.92 | 2.94 | 2.97 | 3.01 | ...... | ...... | ...... | ...... |
| " 7/16 | 8.36 | 1.16 | 2.75 | 2.84 | 2.86 | 2.88 | 2.91 | 2.93 | 2.95 | 2.98 | 3.02 | ...... | ...... | ...... | ...... |
| " ½ | 9.50 | 1.15 | 2.76 | 2.85 | 2.88 | 2.90 | 2.92 | 2.95 | 2.97 | 2.99 | 3.04 | ...... | ...... | ...... | ...... |
| " 9/16 | 10.62 | 1.14 | 2.77 | 2.86 | 2.88 | 2.91 | 2.93 | 2.96 | 2.98 | 3.00 | 3.05 | ...... | ...... | ...... | ...... |
| " ⅝ | 11.72 | 1.13 | 2.78 | 2.87 | 2.89 | 2.92 | 2.94 | 2.97 | 2.99 | 3.01 | 3.06 | ...... | ...... | ...... | ...... |
| " 11/16 | 12.80 | 1.13 | 2.79 | 2.89 | 2.91 | 2.94 | 2.96 | 2.98 | 3.01 | 3.03 | 3.08 | ...... | ...... | ...... | ...... |
| " ¾ | 13.88 | 1.12 | 2.80 | 2.90 | 2.92 | 2.95 | 2.97 | 2.99 | 3.02 | 3.04 | 3.09 | ...... | ...... | ...... | ...... |
| " ⅞ | 15.96 | 1.11 | 2.82 | 2.92 | 2.94 | 2.97 | 2.99 | 3.01 | 3.04 | 3.06 | 3.11 | ...... | ...... | ...... | ...... |
| " 1 | 18.00 | 1.09 | 2.85 | 2.95 | 2.97 | 2.99 | 3.02 | 3.04 | 3.07 | 3.09 | 3.14 | ...... | ...... | ...... | ...... |
| 8x6x7/16 | 11.86 | 1.80 | 3.55 | ...... | ...... | 3.68 | 3.71 | 3.73 | 3.75 | 3.77 | 3.82 | 3.87 | 3.91 | 3.96 | 4.01 |
| " ½ | 13.50 | 1.79 | 3.56 | ...... | ...... | 3.69 | 3.71 | 3.74 | 3.76 | 3.78 | 3.83 | 3.88 | 3.92 | 3.97 | 4.02 |
| " 9/16 | 15.12 | 1.78 | 3.57 | ...... | ...... | 3.71 | 3.73 | 3.75 | 3.77 | 3.80 | 3.84 | 3.89 | 3.94 | 3.99 | 4.03 |
| " ⅝ | 16.72 | 1.77 | 3.58 | ...... | ...... | 3.71 | 3.74 | 3.76 | 3.78 | 3.81 | 3.85 | 3.90 | 3.95 | 4.00 | 4.04 |
| " 11/16 | 18.30 | 1.77 | 3.59 | ...... | ...... | 3.72 | 3.75 | 3.77 | 3.79 | 3.82 | 3.86 | 3.91 | 3.96 | 4.01 | 4.05 |
| " ¾ | 19.88 | 1.76 | 3.60 | ...... | ...... | 3.73 | 3.76 | 3.78 | 3.80 | 3.82 | 3.87 | 3.92 | 3.97 | 4.02 | 4.06 |
| " ⅞ | 22.96 | 1.74 | 3.62 | ...... | ...... | 3.76 | 3.78 | 3.81 | 3.83 | 3.85 | 3.90 | 3.95 | 3.99 | 4.04 | 4.09 |
| " 1 | 26.00 | 1.73 | 3.64 | ...... | ...... | 3.78 | 3.80 | 3.82 | 3.85 | 3.87 | 3.92 | 3.97 | 4.02 | 4.07 | 4.12 |

## TABLE 6.

### Radii of Gyration of Two Angles with Unequal Legs, Both Axes. Short Legs Out.

Radii of Gyration of Two Angles, Short Legs Turned Out.




For Distances Measured from Back to Back.

| Size of Angles. In. | Area Two Angles. In.² | Axis X-X. | Axis Y-Y. Distance Back to Back in Inches. 0 | 1/4 | 5/16 | 3/8 | 1/2 | 5/8 | 3/4 |
|---|---|---|---|---|---|---|---|---|---|
| 2½x2x3/16 | 1.62 | .79 | .79 | .88 | .90 | .92 | .96 | 1.02 | 1.07 |
| " 1/4 | 2.12 | .78 | .80 | .89 | .91 | .93 | .98 | 1.04 | 1.09 |
| " 5/16 | 2.62 | .78 | .81 | .91 | .93 | .95 | 1.00 | 1.05 | 1.10 |
| " 3/8 | 3.10 | .77 | .81 | .92 | .94 | .96 | 1.01 | 1.06 | 1.11 |
| " 7/16 | 3.56 | .76 | .82 | .93 | .95 | .97 | 1.02 | 1.07 | 1.13 |
| 3½x2½x1/4 | 2.88 | 1.12 | .95 | 1.04 | 1.06 | 1.09 | 1.13 | 1.18 | 1.23 |
| " 5/16 | 3.56 | 1.11 | .96 | 1.05 | 1.08 | 1.10 | 1.15 | 1.20 | 1.24 |
| " 3/8 | 4.22 | 1.10 | .97 | 1.07 | 1.09 | 1.11 | 1.16 | 1.21 | 1.26 |
| " 7/16 | 4.86 | 1.09 | .98 | 1.07 | 1.10 | 1.12 | 1.17 | 1.22 | 1.27 |
| " 1/2 | 5.50 | 1.09 | .99 | 1.08 | 1.11 | 1.13 | 1.18 | 1.23 | 1.29 |
| 4x3x1/4 | 3.38 | 1.28 | 1.16 | 1.24 | 1.27 | 1.29 | 1.34 | 1.38 | 1.43 |
| " 5/16 | 4.18 | 1.27 | 1.17 | 1.25 | 1.28 | 1.30 | 1.35 | 1.39 | 1.44 |
| " 3/8 | 4.96 | 1.26 | 1.17 | 1.26 | 1.28 | 1.31 | 1.36 | 1.40 | 1.45 |
| " 7/16 | 5.74 | 1.25 | 1.18 | 1.27 | 1.29 | 1.32 | 1.36 | 1.41 | 1.46 |
| " 1/2 | 6.50 | 1.25 | 1.20 | 1.28 | 1.31 | 1.33 | 1.38 | 1.43 | 1.48 |
| " 9/16 | 7.24 | 1.24 | 1.21 | 1.30 | 1.32 | 1.35 | 1.40 | 1.45 | 1.50 |
| " 5/8 | 7.96 | 1.23 | 1.22 | 1.31 | 1.33 | 1.36 | 1.41 | 1.46 | 1.51 |

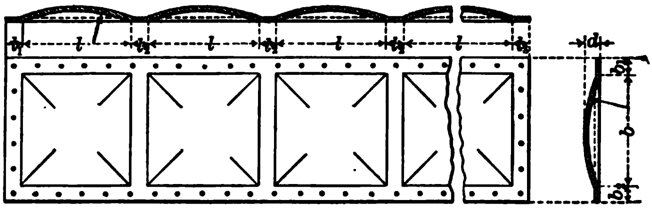
| Size of Angles. In. | Area Two Angles. In.² | Axis X-X. | Axis Y-Y. Distance Back to Back in Inches. 0 | 1/4 | 5/16 | 3/8 | 1/2 | 5/8 | 3/4 |
|---|---|---|---|---|---|---|---|---|---|
| 3x2½x1/4 | 2.62 | .95 | 1.00 | 1.09 | 1.11 | 1.13 | 1.18 | 1.23 | 1.28 |
| " 5/16 | 3.24 | .94 | 1.01 | 1.10 | 1.12 | 1.14 | 1.19 | 1.24 | 1.29 |
| " 3/8 | 3.84 | .93 | 1.02 | 1.11 | 1.14 | 1.16 | 1.21 | 1.26 | 1.31 |
| " 7/16 | 4.44 | .92 | 1.03 | 1.12 | 1.15 | 1.17 | 1.22 | 1.27 | 1.33 |
| " 1/2 | 5.00 | .91 | 1.04 | 1.14 | 1.16 | 1.18 | 1.23 | 1.28 | 1.34 |
| 3½x3x1/4 | 3.12 | 1.11 | 1.20 | 1.29 | 1.31 | 1.33 | 1.38 | 1.43 | 1.48 |
| " 5/16 | 3.86 | 1.10 | 1.22 | 1.30 | 1.32 | 1.35 | 1.39 | 1.44 | 1.49 |
| " 3/8 | 4.60 | 1.09 | 1.23 | 1.31 | 1.33 | 1.36 | 1.40 | 1.45 | 1.50 |
| " 7/16 | 5.30 | 1.08 | 1.23 | 1.32 | 1.34 | 1.37 | 1.41 | 1.46 | 1.51 |
| " 1/2 | 6.00 | 1.07 | 1.24 | 1.33 | 1.36 | 1.39 | 1.43 | 1.48 | 1.53 |
| 5x3x5/16 | 4.80 | 1.61 | 1.09 | 1.17 | 1.20 | 1.22 | 1.26 | 1.31 | 1.36 |
| " 3/8 | 5.72 | 1.61 | 1.09 | 1.18 | 1.21 | 1.23 | 1.27 | 1.32 | 1.37 |
| " 7/16 | 6.62 | 1.60 | 1.10 | 1.20 | 1.22 | 1.24 | 1.29 | 1.34 | 1.39 |
| " 1/2 | 7.50 | 1.59 | 1.11 | 1.21 | 1.23 | 1.25 | 1.30 | 1.35 | 1.40 |
| " 9/16 | 8.36 | 1.58 | 1.12 | 1.22 | 1.24 | 1.26 | 1.31 | 1.36 | 1.41 |
| " 5/8 | 9.22 | 1.57 | 1.14 | 1.23 | 1.26 | 1.28 | 1.33 | 1.38 | 1.43 |
| " 11/16 | 10.06 | 1.56 | 1.15 | 1.24 | 1.27 | 1.29 | 1.34 | 1.39 | 1.44 |

| Size of Angles. In. | Area Two Angles. In.² | Axis X-X. | Axis Y-Y. Distance Back to Back of Angles in Inches. 0 | 1/4 | 5/16 | 3/8 | 7/16 | 1/2 | 9/16 | 5/8 | 3/4 | 7/8 | 1 | 1⅛ | 1¼ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 5x3½x5/16 | 5.12 | 1.61 | 1.33 | 1.41 | 1.43 | 1.46 | 1.48 | 1.50 | 1.52 | 1.55 | 1.59 | ...... | ...... | ...... | ...... |
| " 3/8 | 6.10 | 1.60 | 1.34 | 1.42 | 1.44 | 1.46 | 1.49 | 1.51 | 1.53 | 1.56 | 1.60 | ...... | ...... | ...... | ...... |
| " 7/16 | 7.06 | 1.59 | 1.35 | 1.43 | 1.45 | 1.47 | 1.50 | 1.52 | 1.54 | 1.57 | 1.62 | ...... | ...... | ...... | ...... |
| " 1/2 | 8.00 | 1.58 | 1.36 | 1.44 | 1.47 | 1.49 | 1.51 | 1.54 | 1.56 | 1.58 | 1.63 | ...... | ...... | ...... | ...... |
| " 9/16 | 8.94 | 1.57 | 1.37 | 1.45 | 1.48 | 1.50 | 1.52 | 1.55 | 1.57 | 1.59 | 1.64 | ...... | ...... | ...... | ...... |
| " 5/8 | 9.84 | 1.56 | 1.38 | 1.46 | 1.49 | 1.51 | 1.53 | 1.56 | 1.58 | 1.60 | 1.66 | ...... | ...... | ...... | ...... |
| " 11/16 | 10.74 | 1.56 | 1.38 | 1.47 | 1.50 | 1.52 | 1.54 | 1.57 | 1.59 | 1.62 | 1.67 | ...... | ...... | ...... | ...... |
| " 3/4 | 11.62 | 1.55 | 1.40 | 1.49 | 1.51 | 1.54 | 1.56 | 1.59 | 1.61 | 1.63 | 1.69 | ...... | ...... | ...... | ...... |
| 6x4x3/8 | 7.22 | 1.93 | 1.50 | 1.58 | 1.60 | 1.62 | 1.64 | 1.66 | 1.69 | 1.71 | 1.76 | ...... | ...... | ...... | ...... |
| " 7/16 | 8.36 | 1.92 | 1.50 | 1.59 | 1.61 | 1.63 | 1.66 | 1.68 | 1.70 | 1.72 | 1.77 | ...... | ...... | ...... | ...... |
| " 1/2 | 9.50 | 1.91 | 1.51 | 1.60 | 1.62 | 1.65 | 1.67 | 1.69 | 1.71 | 1.74 | 1.78 | ...... | ...... | ...... | ...... |
| " 9/16 | 10.62 | 1.90 | 1.52 | 1.61 | 1.63 | 1.66 | 1.68 | 1.70 | 1.72 | 1.75 | 1.79 | ...... | ...... | ...... | ...... |
| " 5/8 | 11.72 | 1.90 | 1.53 | 1.62 | 1.64 | 1.67 | 1.69 | 1.71 | 1.73 | 1.76 | 1.81 | ...... | ...... | ...... | ...... |
| " 11/16 | 12.80 | 1.89 | 1.55 | 1.63 | 1.66 | 1.68 | 1.71 | 1.73 | 1.75 | 1.77 | 1.82 | ...... | ...... | ...... | ...... |
| " 3/4 | 13.88 | 1.88 | 1.56 | 1.64 | 1.67 | 1.69 | 1.72 | 1.74 | 1.76 | 1.79 | 1.84 | ...... | ...... | ...... | ...... |
| " 7/8 | 15.96 | 1.86 | 1.58 | 1.66 | 1.69 | 1.71 | 1.74 | 1.76 | 1.79 | 1.81 | 1.86 | ...... | ...... | ...... | ...... |
| " 1 | 18.00 | 1.85 | 1.60 | 1.69 | 1.72 | 1.74 | 1.77 | 1.79 | 1.82 | 1.84 | 1.89 | ...... | ...... | ...... | ...... |
| 8x6x7/16 | 11.86 | 2.57 | 2.31 | ...... | ...... | 2.43 | 2.45 | 2.47 | 2.49 | 2.52 | 2.56 | 2.61 | 2.66 | 2.70 | 2.75 |
| " 1/2 | 13.50 | 2.56 | 2.32 | ...... | ...... | 2.44 | 2.46 | 2.48 | 2.51 | 2.53 | 2.57 | 2.62 | 2.66 | 2.71 | 2.76 |
| " 9/16 | 15.12 | 2.55 | 2.33 | ...... | ...... | 2.46 | 2.48 | 2.50 | 2.52 | 2.54 | 2.59 | 2.63 | 2.68 | 2.73 | 2.77 |
| " 5/8 | 16.72 | 2.54 | 2.34 | ...... | ...... | 2.46 | 2.49 | 2.51 | 2.53 | 2.55 | 2.60 | 2.64 | 2.69 | 2.74 | 2.79 |
| " 11/16 | 18.30 | 2.54 | 2.34 | ...... | ...... | 2.47 | 2.49 | 2.52 | 2.54 | 2.56 | 2.61 | 2.65 | 2.70 | 2.75 | 2.80 |
| " 3/4 | 19.88 | 2.53 | 2.35 | ...... | ...... | 2.48 | 2.50 | 2.52 | 2.55 | 2.57 | 2.62 | 2.66 | 2.71 | 2.77 | 2.81 |
| " 7/8 | 22.96 | 2.51 | 2.37 | ...... | ...... | 2.51 | 2.53 | 2.55 | 2.57 | 2.59 | 2.64 | 2.69 | 2.74 | 2.79 | 2.83 |
| " 1 | 26.00 | 2.49 | 2.39 | ...... | ...... | 2.52 | 2.54 | 2.57 | 2.59 | 2.62 | 2.66 | 2.71 | 2.76 | 2.81 | 2.86 |

## TABLE 7.

### Buckle Plates.

**AMERICAN BRIDGE COMPANY STANDARD.**



| Die Number. | Size of Buckle. | | Rise d, In. | Radii of Buckle. | | Number of Buckles in One Plate. | Widths of Flanges and Fillets. | | |
|---|---|---|---|---|---|---|---|---|---|
| | Side l, Ft.-In. | Side b, Ft.-In. | | Side l, Ft.-In. | Side b, Ft.-In. | | End Flanges $l_1$, $l_3$. | Fillets $l_2$. | Side Flanges $b_1$, $b_2$. |
| 1 | 3–11 | 4– 6 | $3\frac{1}{2}$ | 6– $8\frac{5}{8}$ | 8– $9\frac{7}{8}$ | 1 to 8 | Minimum = 2″——Maximum = 1′–6″ Preferably made alike. If wider than 1′–6″ use angles riveted across the plate for stiffeners | Minimum = 2″——Maximum = 6″ 4″ or less preferred | Minimum = 2″——Maximum = $6\frac{1}{4}$″ Preferably made alike. *Note.*—When the side flanges $b_1$ and $b_2$ are of unequal width, the material should be ordered wide enough to make two flanges of the greater width, the narrower flange to be sheared to required width after buckling. |
| 2 | 4– 6 | 3–11 | $3\frac{1}{2}$ | 8– $9\frac{7}{8}$ | 6– $8\frac{5}{8}$ | 1 to 7 | | | |
| 3 | 3–11 | 3– 6 | 3 | 7– $9\frac{1}{2}$ | 6– 3 | 1 to 8 | | | |
| 4 | 3– 6 | 3–11 | 3 | 6– 3 | 7– $9\frac{1}{2}$ | 1 to 9 | | | |
| 5 | 3– 9 | 3– 9 | 3 | 7– $1\frac{7}{8}$ | 7– $1\frac{7}{8}$ | 1 to 8 | | | |
| 6 | 3– 1 | 3– 9 | 3 | 4–$10\frac{5}{8}$ | 7– $1\frac{7}{8}$ | 1 to 10 | | | |
| 7 | 3– 9 | 3– 1 | 3 | 7– $1\frac{7}{8}$ | 4–$10\frac{5}{8}$ | 1 to 8 | | | |
| 8 | 3– 8 | 3– 8 | 2 | 10– 2 | 10– 2 | 1 to 8 | | | |
| 9 | 2– 8 | 3– 8 | 2 | 5– 5 | 10– 2 | 1 to 11 | | | |
| 10 | 3– 8 | 2– 8 | 2 | 10– 2 | 5– 5 | 1 to 8 | | | |
| 11 | 2– 2 | 3– 8 | 2 | 3– $7\frac{1}{4}$ | 10– 2 | 1 to 14 | | | |
| 12 | 3– 8 | 2– 2 | 2 | 10– 2 | 3– $7\frac{1}{4}$ | 1 to 8 | | | |
| 13 | 3– 0 | 3– 0 | 2 | 6–10 | 6–10 | 1 to 10 | | | |
| 14 | 2– 9 | 2– 9 | 3 | 3–$10\frac{7}{8}$ | 3–$10\frac{7}{8}$ | 1 to 11 | | | |
| 19 | 2– 6 | 2– 9 | $2\frac{1}{2}$ | 3–$10\frac{1}{4}$ | 4– $7\frac{7}{8}$ | 1 to 12 | | | |
| 20 | 2– 9 | 2– 6 | $2\frac{1}{2}$ | 4– $7\frac{7}{8}$ | 3–$10\frac{1}{4}$ | 1 to 11 | | | |
| 21 | 2– 6 | 2– 6 | $2\frac{1}{2}$ | 3–$10\frac{1}{4}$ | 3–$10\frac{1}{4}$ | 1 to 12 | | | |
| 22 | 3– 5 | 3– 6 | 3 | 5–$11\frac{9}{16}$ | 6– 3 | 1 to 9 | | | |
| 23 | 3– 6 | 3– 5 | 3 | 6– 3 | 5–$11\frac{9}{16}$ | 1 to 9 | | | |
| 24 | 3– 6 | 3– 9 | 3 | 6– 3 | 7– $1\frac{7}{8}$ | 1 to 9 | | | |
| 25 | 3– 9 | 3– 6 | 3 | 7– $1\frac{7}{8}$ | 6– 3 | 1 to 8 | | | |
| 26 | 3– 2 | 3– 1 | 3 | 5– $1\frac{21}{32}$ | 4–$10\frac{5}{8}$ | 1 to 9 | | | |
| 27 | 3– 1 | 3– 2 | 3 | 4–$10\frac{5}{8}$ | 5– $1\frac{21}{32}$ | 1 to 10 | | | |
| 28 | 3– 0 | 3– 1 | 3 | 4– $7\frac{1}{2}$ | 4–$10\frac{5}{8}$ | 1 to 10 | | | |
| 29 | 3– 1 | 3– 0 | 3 | 4–$10\frac{5}{8}$ | 4– $7\frac{1}{2}$ | 1 to 10 | | | |
| 30 | 2– 6 | 2– 0 | $2\frac{1}{2}$ | 3–$10\frac{1}{4}$ | 2– $6\frac{1}{16}$ | 1 to 12 | | | |
| 31 | 2– 0 | 2– 6 | $2\frac{1}{2}$ | 2– $6\frac{1}{16}$ | 3–$10\frac{1}{4}$ | 1 to 15 | | | |
| 32 | 5– 6 | 3– 6 | $3\frac{1}{2}$ | 13– $1\frac{21}{64}$ | 5– $4\frac{1}{4}$ | 1 to 5 | | | |
| 33 | 3– 6 | 5– 6 | $3\frac{1}{2}$ | 5– $4\frac{1}{4}$ | 13– $1\frac{21}{64}$ | 1 to 9 | | | |
| 34 | 4– 0 | 4– 0 | 3 | 8– $1\frac{1}{2}$ | 8– $1\frac{1}{2}$ | 1 to 7 | | | |

Plates are steel $\frac{1}{4}$″, $\frac{5}{16}$″, $\frac{3}{8}$″ or $\frac{7}{16}$″ thick.

Plates of greater length than given in table may be made by splicing with bars, angles, or tees.

All plates are made with buckles up, unless otherwise ordered. When buckles are turned down, a drain hole should be punched in the center of each buckle and should be shown on sketch.

Buckles of different sizes should not be used as it increases the cost of the plate.

Connection holes are generally for $\frac{5}{8}$″, $\frac{3}{4}$″ or $\frac{7}{8}$″ rivets or bolts. Different sized holes in same plate will increase the cost of the plate.

Spacing for holes lengthwise of plate should be in multiples of 3″ and should not exceed 12″. Odd spaces to be at end of plate and in even $\frac{1}{4}$″. Minimum spacing crosswise $4\frac{1}{2}$″, usually 6″.

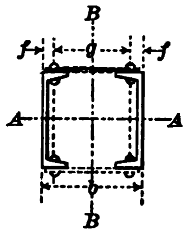
Die number must be shown on drawings.

Sketches for Buckle Plates should indicate allowable overrun in length and width.

## TABLE 8.

### Properties of Column Sections.

Properties of Two Channels Laced.




Flanges Turned In.

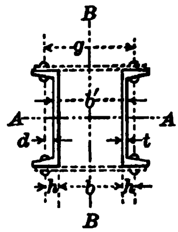
| Channels. | | Total Area. | Moments of Inertia and Radii of Gyration. | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Axis A-A. | | Axis B-B. | | | | | | | | | |
| | | | | | Distance Back to Back of Channels in Inches = b. | | | | | | | | | |
| | | | | | 7½ | | 8½ | | 9½ | | 10½ | | 11½ | |
| Depth. | Wt. | | $I_A$ | $r_A$ | $I_B$ | $r_B$ | $I_B$ | $r_B$ | $I_B$ | $r_B$ | $I_B$ | $r_B$ | $I_B$ | $r_B$ |
| In. | Lb. | In.² | In.⁴ | In. | In.⁴ | In. | In.⁴ | In. | In.⁴ | In. | In.⁴ | In. | In.⁴ | In. |
| 7 | 9.75 | 5.70 | 42.2 | 2.72 | 60.5 | 3.26 | 80.2 | 3.75 | 102.7 | 4.24 | 128.1 | 4.74 | 156.3 | 5.24 |
| " | 12.25 | 7.20 | 48.4 | 2.59 | 77.1 | 3.27 | 102.1 | 3.77 | 130.7 | 4.26 | 162.9 | 4.76 | 198.7 | 5.27 |
| | | | | | 7½ | | 8½ | | 9½ | | 10½ | | 11½ | |
| 8 | 11.25 | 6.70 | 64.6 | 3.10 | 70.2 | 3.24 | 93.1 | 3.73 | 119.4 | 4.22 | 149.0 | 4.72 | 182.0 | 5.21 |
| " | 13.75 | 8.08 | 72.0 | 2.98 | 85.5 | 3.25 | 113.3 | 3.74 | 145.2 | 4.23 | 181.1 | 4.73 | 221.0 | 5.23 |
| | | | | | 8½ | | 9½ | | 10½ | | 11½ | | 12½ | |
| 9 | 13.25 | 7.78 | 94.6 | 3.49 | 106.8 | 3.70 | 137.1 | 4.20 | 171.2 | 4.69 | 209.3 | 5.18 | 251.3 | 5.68 |
| " | 15.00 | 8.82 | 101.8 | 3.40 | 122.0 | 3.72 | 156.5 | 4.21 | 195.4 | 4.71 | 238.7 | 5.20 | 286.4 | 5.70 |
| " | 20.00 | 11.76 | 121.6 | 3.21 | 162.9 | 3.72 | 208.9 | 4.22 | 260.8 | 4.71 | 318.6 | 5.20 | 382.3 | 5.70 |
| | | | | | 9½ | | 10½ | | 11½ | | 12½ | | 13½ | |
| 10 | 15.00 | 8.92 | 133.8 | 3.87 | 155.3 | 4.17 | 194.2 | 4.68 | 237.6 | 5.16 | 285.4 | 5.66 | 337.7 | 6.15 |
| " | 20.00 | 11.76 | 157.4 | 3.66 | 207.4 | 4.20 | 259.0 | 4.69 | 316.5 | 5.19 | 379.9 | 5.68 | 449.2 | 6.18 |
| " | 25.00 | 14.70 | 182.0 | 3.52 | 257.5 | 4.18 | 321.9 | 4.68 | 393.7 | 5.18 | 472.8 | 5.67 | 559.2 | 6.17 |
| | | | | | 10½ | | 11½ | | 12½ | | 13½ | | 14½ | |
| 12 | 20.50 | 12.06 | 256.2 | 4.61 | 257.1 | 4.62 | 314.9 | 5.11 | 378.8 | 5.59 | 448.7 | 6.10 | 524.6 | 6.59 |
| " | 25.00 | 14.70 | 288.0 | 4.43 | 316.3 | 4.64 | 387.2 | 5.13 | 465.4 | 5.62 | 551.0 | 6.12 | 644.0 | 6.62 |
| " | 30.00 | 17.64 | 323.4 | 4.28 | 379.3 | 4.63 | 464.4 | 5.13 | 558.3 | 5.63 | 661.0 | 6.12 | 772.5 | 6.62 |
| " | 35.00 | 20.58 | 358.6 | 4.17 | 439.0 | 4.62 | 537.9 | 5.12 | 647.1 | 5.61 | 766.6 | 6.10 | 896.4 | 6.60 |
| | | | | | 12½ | | 13½ | | 14½ | | 15½ | | 16½ | |
| 15 | 33.00 | 19.80 | 625.2 | 5.62 | 605.9 | 5.53 | 718.9 | 6.02 | 841.7 | 6.52 | 974.5 | 7.02 | 1117.2 | 7.51 |
| " | 35.00 | 20.58 | 640.0 | 5.57 | 630.7 | 5.54 | 748.2 | 6.03 | 876.0 | 6.52 | 1014.2 | 7.02 | 1162.6 | 7.52 |
| " | 40.00 | 23.52 | 695.0 | 5.44 | 721.7 | 5.54 | 856.2 | 6.03 | 1002.4 | 6.51 | 1160.4 | 7.03 | 1330.2 | 7.52 |
| " | 45.00 | 26.48 | 750.2 | 5.32 | 810.6 | 5.53 | 961.9 | 6.02 | 1126.4 | 6.52 | 1304.1 | 7.02 | 1495.1 | 7.52 |

The table given above is intended to serve only as a guide in the choice of sections, and not as a complete table. For additional values, see Ketchum's "Structural Engineers' Handbook."

# TABLE 9.

## Properties of Column Sections.

Properties of Two Channels Laced.




Flanges Turned Out.

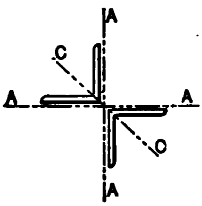
| Channels. | | Total Area. | Moments of Inertia and Radii of Gyration. | | | | | | | | Web of Channel. | Gages. | | Max. Rivet. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Axis A–A. | | Axis B–B. Distance Inside to Inside of Webs in Inches = b'. | | | | | | | | | |
| Depth. | Weight. | | | | $4\frac{3}{4}$ | | $5\frac{3}{4}$ | | $6\frac{3}{4}$ | | | | | |
| | | | $I_A$ | $r_A$ | $I_B$ | $r_B$ | $I_B$ | $r_B$ | $I_B$ | $r_B$ | t | g | h | |
| In. | Lb. | In.$^2$ | In.$^4$ | In. | In.$^4$ | In. | In.$^4$ | In. | In.$^4$ | In. | In. | In. | In. | In. |
| 7 | 9.75 | 5.70 | 42 | 2.72 | 43 | 2.73 | 59 | 3.22 | 79 | 3.72 | $\frac{1}{4}$ | 1 | $1\frac{1}{4}$ | $\frac{5}{8}$ |
| " | 12.25 | 7.20 | 48 | 2.59 | 51 | 2.65 | 71 | 3.14 | 95 | 3.64 | $\frac{5}{16}$ | " | $1\frac{5}{16}$ | " |
| | | | | | $4\frac{1}{2}$ | | $5\frac{1}{2}$ | | $6\frac{1}{2}$ | | | | | |
| 8 | 11.25 | 6.70 | 65 | 3.10 | 47 | 2.65 | 66 | 3.14 | 88 | 3.63 | $\frac{1}{4}$ | 1 | $1\frac{1}{4}$ | $\frac{3}{4}$ |
| " | 13.75 | 8.08 | 72 | 2.98 | 53 | 2.57 | 76 | 3.06 | 102 | 3.55 | $\frac{5}{16}$ | " | $1\frac{5}{16}$ | " |
| " | 16.25 | 9.56 | 80 | 2.89 | 57 | 2.45 | 82 | 2.94 | 112 | 3.43 | $\frac{7}{16}$ | " | $1\frac{7}{16}$ | " |
| | | | | | $6\frac{1}{2}$ | | $7\frac{1}{2}$ | | $8\frac{1}{2}$ | | | | | |
| 9 | 13.25 | 7.78 | 95 | 3.49 | 98 | 3.55 | 127 | 4.04 | 160 | 4.54 | $\frac{1}{4}$ | $1\frac{1}{8}$ | $1\frac{3}{8}$ | $\frac{3}{4}$ |
| " | 15.00 | 8.82 | 102 | 3.40 | 106 | 3.47 | 138 | 3.95 | 175 | 4.45 | $\frac{5}{16}$ | " | $1\frac{7}{16}$ | " |
| " | 20.00 | 11.76 | 122 | 3.21 | 131 | 3.34 | 172 | 3.83 | 220 | 4.32 | $\frac{7}{16}$ | " | $1\frac{9}{16}$ | " |
| | | | | | 6 | | 7 | | 8 | | | | | |
| 10 | 15.00 | 8.92 | 134 | 3.87 | 107 | 3.46 | 140 | 3.95 | 176 | 4.44 | $\frac{1}{4}$ | $1\frac{1}{4}$ | $1\frac{1}{2}$ | $\frac{3}{4}$ |
| " | 20.00 | 11.76 | 157 | 3.66 | 129 | 3.31 | 170 | 3.80 | 217 | 4.29 | $\frac{3}{8}$ | " | $1\frac{5}{8}$ | " |
| " | 25.00 | 14.70 | 182 | 3.52 | 150 | 3.19 | 199 | 3.68 | 256 | 4.17 | $\frac{1}{2}$ | " | $1\frac{3}{4}$ | " |
| | | | | | 8 | | 9 | | 10 | | | | | |
| 12 | 20.50 | 12.06 | 256 | 4.61 | 240 | 4.47 | 296 | 4.96 | 358 | 5.45 | $\frac{5}{16}$ | $1\frac{1}{2}$ | $1\frac{13}{16}$ | $\frac{7}{8}$ |
| " | 25.00 | 14.70 | 288 | 4.43 | 281 | 4.37 | 348 | 4.87 | 423 | 5.36 | $\frac{3}{8}$ | " | $1\frac{7}{8}$ | " |
| " | 35.00 | 20.58 | 359 | 4.17 | 353 | 4.14 | 441 | 4.63 | 541 | 5.13 | $\frac{5}{8}$ | " | $2\frac{1}{8}$ | " |
| | | | | | $9\frac{1}{2}$ | | $10\frac{1}{2}$ | | $11\frac{1}{2}$ | | | | | |
| 15 | 33.00 | 19.80 | 625 | 5.62 | 540 | 5.22 | 646 | 5.68 | 763 | 6.18 | $\frac{7}{16}$ | $1\frac{3}{4}$ | $2\frac{3}{16}$ | $\frac{7}{8}$ |
| " | 45.00 | 26.48 | 750 | 5.32 | 660 | 4.99 | 796 | 5.48 | 946 | 5.98 | $\frac{5}{8}$ | " | $2\frac{3}{8}$ | " |
| " | 55.00 | 32.36 | 860 | 5.16 | 758 | 4.84 | 920 | 5.33 | 1098 | 5.83 | $\frac{13}{16}$ | " | $2\frac{9}{16}$ | " |

The table given above is intended to serve only as a guide in the choice of sections, and not as a complete table. For additional values, see Ketchum's "Structural Engineers' Handbook."

## TABLE 10.

### PROPERTIES OF STARRED ANGLES.

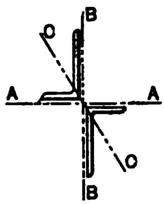
**Two Angles Starred, Equal Legs.**




Values for Axis A–A same as in Table 4.

| Size of Angles. | Total Area. | Least Radius of Gyration. |
|---|---|---|
| | A | $r_C$ |
| In. | In.² | In. |
| 2x2x¼ | 1.88 | .77 |
| " ⅜ | 2.72 | .74 |
| 2½x2½x¼ | 2.38 | .97 |
| " ⅜ | 3.46 | .95 |
| 3x3x¼ | 2.88 | 1.17 |
| " ⅜ | 4.22 | 1.16 |
| " ½ | 5.50 | 1.13 |
| " ⅝ | 6.72 | 1.10 |
| 3½x3½x¼ | 3.38 | 1.37 |
| " ⅜ | 4.96 | 1.35 |
| " ½ | 6.50 | 1.33 |
| " ⅝ | 7.96 | 1.31 |
| 4x4x¼ | 3.88 | 1.58 |
| " ⅜ | 5.72 | 1.56 |
| " ½ | 7.50 | 1.53 |
| " ⅝ | 9.22 | 1.51 |
| 5x5x⅜ | 7.22 | 1.98 |
| " ½ | 9.50 | 1.95 |
| " ⅝ | 11.72 | 1.92 |
| " ¾ | 13.88 | 1.89 |
| 6x6x⅜ | 8.72 | 2.37 |
| " ½ | 11.50 | 2.35 |
| " ⅝ | 14.22 | 2.33 |
| " ¾ | 16.88 | 2.30 |
| " ⅞ | 19.46 | 2.28 |
| " 1 | 22.00 | 2.26 |
| 8x8x½ | 15.50 | 3.17 |
| " ⅝ | 19.22 | 3.14 |
| " ¾ | 22.88 | 3.12 |
| " ⅞ | 26.46 | 3.09 |
| " 1 | 30.00 | 3.07 |

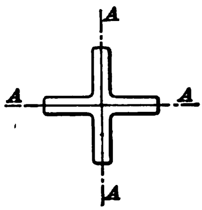
**Two Angles Starred, Unequal Legs.**




Values for Axes A–A & B–B same as in Tables 5 & 6, respectively.

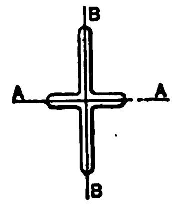
| Size of Angles. | Total Area. | Least Radius of Gyration. |
|---|---|---|
| | A | $r_C$ |
| In. | In.² | In. |
| 2½x2x¼ | 2.12 | .73 |
| " ⅜ | 3.10 | .78 |
| 3x2½x¼ | 2.62 | 1.00 |
| " ⅜ | 3.84 | 1.00 |
| 3½x3x¼ | 3.12 | 1.22 |
| " ⅜ | 4.60 | 1.20 |
| " ½ | 6.00 | 1.18 |
| " ⅝ | 7.34 | 1.16 |
| 4x3x¼ | 3.38 | 1.23 |
| " ⅜ | 4.96 | 1.21 |
| " ½ | 6.50 | 1.19 |
| " ⅝ | 7.96 | 1.17 |
| 5x3x⅜ | 5.72 | 1.16 |
| " ½ | 7.50 | 1.16 |
| " ⅝ | 9.22 | 1.15 |
| " ¾ | 10.88 | 1.15 |
| 5x3½x⅜ | 6.10 | 1.37 |
| " ½ | 8.00 | 1.35 |
| " ⅝ | 9.84 | 1.34 |
| " ¾ | 11.62 | 1.33 |
| 6x4x⅜ | 7.22 | 1.56 |
| " ½ | 9.50 | 1.56 |
| " ⅝ | 11.72 | 1.55 |
| " ¾ | 13.88 | 1.55 |
| " ⅞ | 15.96 | 1.54 |
| " 1 | 18.00 | 1.54 |
| 8x6x½ | 13.50 | 2.39 |
| " ⅝ | 16.72 | 2.38 |
| " ¾ | 19.88 | 2.36 |
| " ⅞ | 22.96 | 2.35 |
| " 1 | 26.00 | 2.34 |

**Four Angles Starred, Equal Legs.**




| Size of Angles. | Total Area. | Radius of Gyration. |
|---|---|---|
| | A | $r_A$ |
| In. | In.² | In. |
| 2x2x¼ | 3.76 | .85 |
| " ⅜ | 5.44 | .88 |
| 2½x2½x¼ | 4.76 | 1.05 |
| " ⅜ | 6.92 | 1.07 |
| 3x3x¼ | 5.76 | 1.25 |
| " ⅜ | 8.44 | 1.27 |
| " ½ | 11.00 | 1.29 |
| " ⅝ | 13.44 | 1.32 |
| 3½x3½x¼ | 6.76 | 1.45 |
| " ⅜ | 9.92 | 1.48 |
| " ½ | 13.00 | 1.50 |
| " ⅝ | 15.92 | 1.52 |
| 4x4x¼ | 7.76 | 1.66 |
| " ⅜ | 11.44 | 1.68 |
| " ½ | 15.00 | 1.70 |
| " ⅝ | 18.44 | 1.72 |
| 5x5x⅜ | 14.44 | 2.08 |
| " ½ | 19.00 | 2.10 |
| " ⅝ | 23.44 | 2.12 |
| " ¾ | 27.76 | 2.14 |
| 6x6x⅜ | 17.44 | 2.49 |
| " ½ | 23.00 | 2.51 |
| " ⅝ | 28.44 | 2.53 |
| " ¾ | 33.76 | 2.55 |
| " ⅞ | 38.92 | 2.57 |
| " 1 | 44.00 | 2.59 |
| 8x8x½ | 31.00 | 3.32 |
| " ⅝ | 38.44 | 3.34 |
| " ¾ | 45.76 | 3.36 |
| " ⅞ | 52.92 | 3.38 |
| " 1 | 60.00 | 3.40 |

**Four Angles Starred, Unequal Legs.**




| Size of Angles. | Total Area. | Radius of Gyration. Axis A–A. | Radius of Gyration. Axis B–B. |
|---|---|---|---|
| | A | $r_A$ | $r_B$ |
| In. | In.² | In. | In. |
| 2½x2x¼ | 4.24 | 1.11 | .80 |
| " ⅜ | 6.20 | 1.13 | .81 |
| 3x2½x¼ | 5.24 | 1.31 | 1.00 |
| " ⅜ | 7.68 | 1.33 | 1.02 |
| 3½x3x¼ | 6.24 | 1.52 | 1.20 |
| " ⅜ | 9.20 | 1.53 | 1.23 |
| " ½ | 12.00 | 1.55 | 1.24 |
| " ⅝ | 14.68 | 1.57 | 1.26 |
| 4x3x¼ | 6.76 | 1.77 | 1.16 |
| " ⅜ | 9.92 | 1.80 | 1.17 |
| " ½ | 13.00 | 1.82 | 1.20 |
| " ⅝ | 15.92 | 1.84 | 1.22 |
| 5x3x⅜ | 11.44 | 2.34 | 1.09 |
| " ½ | 15.00 | 2.36 | 1.11 |
| " ⅝ | 18.44 | 2.39 | 1.14 |
| " ¾ | 21.76 | 2.41 | 1.16 |
| 5x3½x⅜ | 12.20 | 2.27 | 1.34 |
| " ½ | 16.00 | 2.29 | 1.36 |
| " ⅝ | 19.68 | 2.31 | 1.38 |
| " ¾ | 23.24 | 2.33 | 1.40 |
| 6x4x⅜ | 14.44 | 2.74 | 1.50 |
| " ½ | 19.00 | 2.76 | 1.51 |
| " ⅝ | 23.44 | 2.78 | 1.53 |
| " ¾ | 27.76 | 2.80 | 1.56 |
| " ⅞ | 31.92 | 2.82 | 1.58 |
| " 1 | 36.00 | 2.85 | 1.60 |
| 8x6x½ | 27.00 | 3.56 | 2.32 |
| " ⅝ | 33.44 | 3.58 | 2.33 |
| " ¾ | 39.76 | 3.60 | 2.35 |
| " ⅞ | 45.92 | 3.62 | 2.37 |
| " 1 | 52.00 | 3.64 | 2.39 |

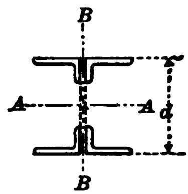
For unequal leg angles, the angle between *B–B* & *C–C* varies between 10° & 34°.
Tie plates for unequal leg angles = ⅜″.

When angles are not in contact, use tables 4, 5, & 6.

## TABLE 11.

### Properties of Four Angles Laced.

Properties of Four Angles Laced.




For Equal Legs and Unequal Legs with Long Legs Turned Out.

| Four Angles. | Total Area. | Moments of Inertia and Radii of Gyration. | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Axis B–B. | | | | Axis A–A. | | | | | | | | | |
| | | Thickness of 2 Lacing Bars = t. | | | | Distance Back to Back of Angles in Inches = d. | | | | | | | | | |
| | | 2 Bars ¼″ = ½″. | | 2 Bars 5/16″ = ⅝″. | | 8½ | | 10½ | | 12½ | | 14½ | | 16½ | |
| | | $I_B$ | $r_B$ | $I_B$ | $r_B$ | $I_A$ | $r_A$ | $I_A$ | $r_A$ | $I_A$ | $r_A$ | $I_A$ | $r_A$ | $I_A$ | $r_A$ |
| In. | In.² | In.⁴ | In. | In.⁴ | In. | In.⁴ | In. | In.⁴ | In. | In.⁴ | In. | In.⁴ | In. | In.⁴ | In. |
| 3x2½x¼ | 5.24 | 12 | 1.50 | 13 | 1.55 | 71 | 3.68 | 113 | 4.64 | 167 | 5.64 | 231 | 6.64 | 305 | 7.63 |
| " ⅜ | 7.68 | 18 | 1.53 | 19 | 1.58 | 100 | 3.61 | 162 | 4.59 | 240 | 5.59 | 333 | 6.58 | 440 | 7.58 |
| " ½ | 10.00 | 24 | 1.55 | 26 | 1.60 | 128 | 3.57 | 208 | 4.56 | 308 | 5.55 | 428 | 6.54 | 567 | 7.54 |
| 4x3x⅜ | 9.92 | 39 | 1.98 | 41 | 2.03 | 127 | 3.58 | 206 | 4.56 | 305 | 5.55 | 423 | 6.53 | 561 | 7.52 |
| " ½ | 13.00 | 53 | 2.01 | 55 | 2.06 | 162 | 3.53 | 264 | 4.51 | 392 | 5.49 | 546 | 6.48 | 725 | 7.48 |
| " ⅝ | 15.92 | 66 | 2.04 | 69 | 2.08 | 193 | 3.48 | 317 | 4.46 | 472 | 5.44 | 659 | 6.43 | 879 | 7.42 |
| | | 2 Bars ¼″ = ½″ | | 2 Bars 5/16″ = ⅝″ | | 10½ | | 12½ | | 14½ | | 16½ | | 18½ | |
| 3½x3½x⅜ | 9.92 | 27 | 1.66 | 29 | 1.71 | 190 | 4.38 | 284 | 5.34 | 398 | 6.34 | 532 | 7.32 | 685 | 8.31 |
| " ½ | 13.00 | 37 | 1.69 | 39 | 1.73 | 243 | 4.32 | 365 | 5.30 | 513 | 6.28 | 687 | 7.27 | 887 | 8.26 |
| " ⅝ | 15.92 | 46 | 1.70 | 49 | 1.76 | 291 | 4.27 | 440 | 5.26 | 619 | 6.23 | 831 | 7.18 | 1075 | 8.21 |
| 4x4x⅜ | 11.44 | 39 | 1.86 | 42 | 1.91 | 211 | 4.29 | 316 | 5.25 | 444 | 6.22 | 596 | 7.22 | 770 | 8.20 |
| " ½ | 15.00 | 53 | 1.88 | 56 | 1.93 | 271 | 4.25 | 408 | 5.22 | 575 | 6.19 | 772 | 7.17 | 999 | 8.16 |
| " ⅝ | 18.44 | 67 | 1.91 | 71 | 1.96 | 325 | 4.20 | 491 | 5.16 | 695 | 6.14 | 935 | 7.12 | 1213 | 8.11 |
| | | 2 Bars 5/16″ = ⅝″ | | 2 Bars ⅜″ = ¾″ | | 10½ | | 12½ | | 14½ | | 16½ | | 18½ | |
| 5x3½x⅜ | 12.20 | 76 | 2.50 | 79 | 2.55 | 248 | 4.51 | 367 | 5.48 | 511 | 6.47 | 679 | 7.46 | 872 | 8.45 |
| " ½ | 16.00 | 102 | 2.53 | 106 | 2.58 | 318 | 4.46 | 472 | 5.43 | 659 | 6.41 | 878 | 7.41 | 1129 | 8.40 |
| " ⅝ | 19.68 | 128 | 2.55 | 133 | 2.60 | 382 | 4.40 | 571 | 5.39 | 800 | 6.37 | 1067 | 7.36 | 1374 | 8.36 |
| 6x4x½ | 19.00 | 170 | 2.99 | 176 | 3.04 | 370 | 4.41 | 551 | 5.39 | 770 | 6.36 | 1027 | 7.35 | 1321 | 8.34 |
| " ⅝ | 23.44 | 213 | 3.01 | 220 | 3.06 | 448 | 4.37 | 669 | 5.34 | 937 | 6.32 | 1252 | 7.32 | 1614 | 8.30 |
| " ¾ | 27.76 | 257 | 3.04 | 265 | 3.09 | 517 | 4.32 | 777 | 5.29 | 1092 | 6.27 | 1462 | 7.26 | 1888 | 8.24 |

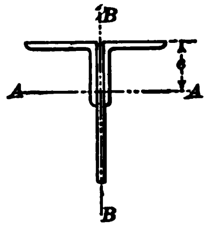
The above table is intended to serve only as a guide in the choice of sections and not as a complete table. For additional values, see Ketchum's "Structural Engineers' Handbook."

## TABLE 12.

### Properties of Chord Sections.

### McClintoc-Marshal Construction Co. Standards.

Properties of Two Angles and One Web Plate.




Long Legs Turned Out. Top of Plate ¼" Below Backs of Angles.

| Size of Web Plate. | Size of Angles. | Total Area. | Axis A-A. Moment of Inertia. | Axis A-A. Radius of Gyration. | Axis A-A. Section Modulus. | Axis A-A. Centroid. | Axis B-B. Moment of Inertia. | Axis B-B. Radius of Gyration. |
|---|---|---|---|---|---|---|---|---|
| | | $A$ | $I_A$ | $r_A$ | $S_A$ | $e$ | $I_B$ | $r_B$ |
| In. | In. | In.$^2$ | In.$^4$ | In. | In.$^3$ | In. | In.$^4$ | In. |
| 6×¼ | 2×2×¼ | 3.38 | 11.1 | 1.81 | 6.3 | 1.77 | 1.7 | .70 |
| | 2½×2×¼ | 3.62 | 11.7 | 1.80 | 7.1 | 1.66 | 3.1 | .93 |
| 7×¼ | 2×2×¼ | 3.63 | 17.1 | 2.17 | 9.1 | 1.87 | 1.7 | .68 |
| | 2½×2×¼ | 3.87 | 17.8 | 2.14 | 8.9 | 1.99 | 3.1 | .90 |
| | 3×2×¼ | 4.13 | 18.7 | 2.13 | 10.0 | 1.87 | 5.1 | 1.12 |
| | 3×2½×¼ | 4.37 | 18.7 | 2.07 | 9.9 | 1.90 | 5.2 | 1.09 |
| 8×¼ | 2×2×¼ | 3.88 | 24.4 | 2.51 | 9.8 | 2.48 | 1.7 | .66 |
| | 2½×2×¼ | 4.12 | 25.6 | 2.49 | 10.9 | 2.34 | 3.1 | .87 |
| | 2½×2½×¼ | 4.38 | 25.6 | 2.42 | 11.0 | 2.33 | 3.1 | .84 |
| | 2½×2½×5/16 | 4.94 | 27.1 | 2.34 | 12.5 | 2.16 | 3.9 | .89 |
| | 3×2×¼ | 4.38 | 26.8 | 2.47 | 12.1 | 2.21 | 5.1 | 1.09 |
| | 3×2½×¼ | 4.62 | 26.8 | 2.41 | 12.1 | 2.22 | 5.2 | 1.06 |
| | 3×2½×5/16 | 5.24 | 28.7 | 2.30 | 13.6 | 2.04 | 6.5 | 1.11 |
| | 3½×2½×¼ | 4.88 | 27.9 | 2.39 | 13.3 | 2.10 | 8.0 | 1.28 |
| | 3½×2½×5/16 | 5.56 | 29.2 | 2.29 | 15.1 | 1.94 | 10.1 | 1.35 |
| 8×5/16 | 2½×2½×5/16 | 5.44 | 31.7 | 2.41 | 13.5 | 2.35 | 4.1 | .87 |
| | 3×2½×¼ | 5.12 | 31.3 | 2.47 | 12.9 | 2.42 | 5.3 | 1.02 |
| | 3×2½×5/16 | 5.74 | 33.2 | 2.40 | 14.8 | 2.24 | 6.7 | 1.08 |
| | 3½×2½×¼ | 5.38 | 32.6 | 2.46 | 14.2 | 2.30 | 8.2 | 1.24 |
| | 3½×2½×5/16 | 6.06 | 34.3 | 2.38 | 16.1 | 2.13 | 10.4 | 1.31 |
| 8×⅜ | 3½×2½×5/16 | 6.56 | 39.1 | 2.44 | 17.1 | 2.29 | 10.6 | 1.28 |
| | 4×3×5/16 | 7.18 | 40.6 | 2.38 | 18.1 | 2.22 | 15.2 | 1.47 |
| | 4×3×⅜ | 7.96 | 42.5 | 2.31 | 20.3 | 2.09 | 18.6 | 1.53 |
| 9×¼ | 2½×2½×¼ | 4.63 | 35.4 | 2.76 | 13.2 | 2.68 | 3.1 | .82 |
| | 3×2×¼ | 4.63 | 37.3 | 2.84 | 13.5 | 2.77 | 5.1 | 1.05 |
| | 3×2½×¼ | 4.87 | 37.0 | 2.75 | 14.5 | 2.55 | 5.2 | 1.03 |
| | 3½×2½×¼ | 5.13 | 38.4 | 2.73 | 15.8 | 2.43 | 8.0 | 1.25 |
| 9×5/16 | 3×2½×5/16 | 6.05 | 45.8 | 2.75 | 17.5 | 2.62 | 6.7 | 1.05 |
| | 3½×2½×5/16 | 6.37 | 47.5 | 2.73 | 19.0 | 2.50 | 10.3 | 1.28 |
| | 3½×2½×⅜ | 7.03 | 49.5 | 2.65 | 21.1 | 2.34 | 12.4 | 1.33 |
| | 4×3×5/16 | 6.99 | 49.1 | 2.65 | 20.2 | 2.41 | 15.1 | 1.48 |

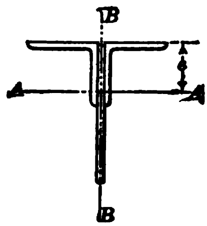
| Size of Web Plate. | Size of Angles. | Total Area. | Axis A-A. Moment of Inertia. | Axis A-A. Radius of Gyration. | Axis A-A. Section Modulus. | Axis A-A. Centroid. | Axis B-B. Moment of Inertia. | Axis B-B. Radius of Gyration. |
|---|---|---|---|---|---|---|---|---|
| | | $A$ | $I_A$ | $r_A$ | $S_A$ | $e$ | $I_B$ | $r_B$ |
| In. | In. | In.$^2$ | In.$^4$ | In. | In.$^3$ | In. | In.$^4$ | In. |
| 10×¼ | 2½×2½×¼ | 4.88 | 47.2 | 3.10 | 15.5 | 3.04 | 3.1 | .80 |
| | 2½×2½×5/16 | 5.44 | 50.1 | 3.03 | 17.8 | 2.82 | 3.9 | .85 |
| | 3×2×¼ | 4.88 | 49.3 | 3.19 | 16.8 | 2.93 | 5.1 | 1.03 |
| | 3×2½×¼ | 5.12 | 49.3 | 3.09 | 17.0 | 2.90 | 5.2 | 1.00 |
| | 3×2½×5/16 | 5.74 | 52.2 | 3.02 | 19.6 | 2.67 | 6.5 | 1.06 |
| | 3½×2½×¼ | 5.38 | 51.3 | 3.09 | 18.5 | 2.77 | 8.0 | 1.22 |
| | 3½×2½×5/16 | 6.06 | 54.0 | 2.99 | 21.2 | 2.55 | 10.1 | 1.29 |
| | 4×3×5/16 | 6.68 | 55.7 | 2.89 | 22.8 | 2.44 | 14.8 | 1.49 |
| 10×5/16 | 2½×2½×5/16 | 6.07 | 58.6 | 3.10 | 19.1 | 3.07 | 4.1 | .82 |
| | 3×2½×¼ | 5.75 | 57.6 | 3.16 | 18.2 | 3.16 | 5.3 | .96 |
| | 3×2½×5/16 | 6.37 | 61.2 | 3.10 | 21.0 | 2.91 | 6.7 | 1.02 |
| | 3½×2½×¼ | 6.01 | 60.0 | 3.16 | 19.8 | 3.03 | 8.2 | 1.17 |
| | 3½×2½×5/16 | 6.69 | 63.4 | 3.08 | 22.7 | 2.80 | 10.3 | 1.25 |
| | 4×3×5/16 | 7.31 | 65.5 | 2.99 | 24.3 | 2.69 | 15.1 | 1.44 |
| | 4×3×⅜ | 8.09 | 68.3 | 2.91 | 27.2 | 2.51 | 18.2 | 1.50 |
| | 5×3×5/16 | 7.93 | 69.2 | 2.96 | 27.8 | 2.49 | 28.7 | 1.91 |
| | 5×3×⅜ | 8.85 | 72.1 | 2.85 | 31.1 | 2.32 | 34.4 | 1.97 |
| | 5×3½×5/16 | 8.25 | 69.3 | 2.89 | 27.6 | 2.51 | 28.8 | 1.87 |
| | 5×3½×⅜ | 9.23 | 72.4 | 2.81 | 30.8 | 2.35 | 34.6 | 1.94 |
| 10×⅜ | 3×2½×5/16 | 6.99 | 69.5 | 3.15 | 22.2 | 3.13 | 6.9 | .99 |
| | 3½×2½×5/16 | 7.31 | 72.1 | 3.14 | 23.9 | 3.01 | 10.6 | 1.21 |
| | 4×3×5/16 | 7.93 | 74.5 | 3.07 | 25.9 | 2.88 | 15.5 | 1.40 |
| | 4×3×⅜ | 8.71 | 77.8 | 2.99 | 28.7 | 2.71 | 18.6 | 1.46 |
| | 5×3×5/16 | 8.55 | 78.9 | 3.03 | 29.3 | 2.69 | 29.7 | 1.85 |
| | 5×3×⅜ | 9.47 | 82.4 | 2.94 | 32.8 | 2.51 | 35.1 | 1.93 |
| 12×¼ | 3×2½×¼ | 5.62 | 81.2 | 3.80 | 22.2 | 3.65 | 5.2 | .96 |
| | 3×2½×5/16 | 6.24 | 86.2 | 3.73 | 25.6 | 3.37 | 6.5 | 1.02 |
| | 3½×2½×¼ | 5.88 | 84.3 | 3.78 | 24.2 | 3.49 | 8.0 | 1.17 |
| | 3½×2½×5/16 | 6.56 | 89.1 | 3.67 | 27.8 | 3.20 | 10.1 | 1.24 |
| | 4×3×5/16 | 7.18 | 92.0 | 3.58 | 30.2 | 3.05 | 14.8 | 1.44 |
| | 5×3×5/16 | 7.80 | 96.8 | 3.52 | 34.3 | 2.82 | 28.1 | 1.90 |
| | 5×3×⅜ | 8.72 | 100.8 | 3.41 | 38.6 | 2.61 | 33.8 | 1.97 |
| | 5×3½×5/16 | 8.12 | 96.8 | 3.45 | 34.0 | 2.85 | 28.3 | 1.87 |
| | 5×3½×⅜ | 9.10 | 100.6 | 3.33 | 38.1 | 2.64 | 33.9 | 1.94 |

## TABLE 12.—*Continued.*

### PROPERTIES OF CHORD SECTIONS.

### McCLINTOC-MARSHALL CONSTRUCTION CO. STANDARDS.

Properties of Two Angles and One Web Plate.




Long Legs Turned Out. Top of Plate ¼″ Below Backs of Angles.

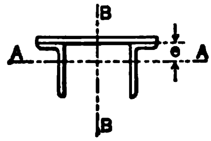
| Size of Web Plate. | Size of Angles. | Total Area. | Axis A–A. Moment of Inertia. | Axis A–A. Radius of Gyration. | Axis A–A. Section Modulus. | Axis A–A. Centroid. | Axis B–B. Moment of Inertia. | Axis B–B. Radius of Gyration. |
|---|---|---|---|---|---|---|---|---|
| | | A | $I_A$ | $r_A$ | $S_A$ | e | $I_B$ | $r_B$ |
| In. | In. | In.² | In.⁴ | In. | In.³ | In. | In.⁴ | In. |
| 12×5/16 | 3 ×2½×¼ | 6.37 | 94.8 | 3.84 | 24.0 | 3.95 | 5.3 | .92 |
| | 3 ×2½×5/16 | 6.99 | 100.7 | 3.79 | 27.5 | 3.67 | 6.7 | .98 |
| | 3½×2½×¼ | 6.63 | 98.5 | 3.86 | 25.9 | 3.81 | 8.2 | 1.11 |
| | 3½×2½×5/16 | 7.31 | 104.5 | 3.78 | 29.6 | 3.53 | 10.3 | 1.19 |
| | 4 ×3 ×5/16 | 7.93 | 107.9 | 3.70 | 32.0 | 3.37 | 15.1 | 1.38 |
| | 4 ×3 ×3/8 | 8.71 | 112.8 | 3.60 | 35.8 | 3.15 | 18.2 | 1.44 |
| | 5 ×3 ×5/16 | 8.55 | 113.8 | 3.64 | 36.3 | 3.13 | 28.7 | 1.82 |
| | 5 ×3 ×3/8 | 9.47 | 119.0 | 3.55 | 40.9 | 2.91 | 34.4 | 1.92 |
| | 5 ×3½×5/16 | 8.87 | 113.9 | 3.58 | 36.4 | 3.13 | 28.8 | 1.80 |
| | 5 ×3½×3/8 | 9.85 | 119.0 | 3.47 | 40.9 | 2.92 | 34.6 | 1.88 |
| 12×3/8 | 3 ×2½×5/16 | 7.74 | 114.3 | 3.84 | 29.3 | 3.91 | 6.9 | .95 |
| | 3½×2½×5/16 | 8.06 | 118.5 | 3.83 | 31.4 | 3.77 | 10.6 | 1.15 |
| | 4 ×3 ×5/16 | 8.68 | 122.7 | 3.76 | 34.0 | 3.61 | 15.5 | 1.34 |
| | 4 ×3 ×3/8 | 9.46 | 128.4 | 3.68 | 38.0 | 3.38 | 18.6 | 1.40 |
| | 5 ×3 ×5/16 | 9.30 | 129.9 | 3.74 | 38.4 | 3.38 | 29.2 | 1.77 |
| | 5 ×3 ×3/8 | 10.22 | 135.8 | 3.64 | 43.2 | 3.14 | 35.1 | 1.85 |
| | 5 ×3½×5/16 | 9.62 | 129.5 | 3.80 | 38.4 | 3.37 | 29.4 | 1.75 |
| | 5 ×3½×3/8 | 10.60 | 135.8 | 3.58 | 43.1 | 3.15 | 35.3 | 1.82 |
| | 6 ×3½×3/8 | 11.34 | 141.8 | 3.54 | 47.9 | 2.96 | 59.6 | 2.30 |
| | 6 ×4 ×3/8 | 11.72 | 145.0 | 3.52 | 48.7 | 2.98 | 59.6 | 2.26 |
| 12×7/16 | 4 ×3 ×7/16 | 10.99 | 149.1 | 3.68 | 45.1 | 3.31 | 22.3 | 1.42 |
| | 5 ×3 ×3/8 | 10.97 | 150.0 | 3.69 | 44.8 | 3.35 | 35.8 | 1.81 |
| | 5 ×3½×3/8 | 11.35 | 151.5 | 3.65 | 45.2 | 3.35 | 35.9 | 1.78 |
| | 5 ×3½×7/16 | 12.31 | 157.1 | 3.57 | 49.6 | 3.17 | 42.0 | 1.85 |
| | 6 ×3½×3/8 | 12.09 | 158.4 | 3.62 | 50.2 | 3.16 | 60.6 | 2.24 |
| | 6 ×3½×7/16 | 13.19 | 164.3 | 3.52 | 55.0 | 2.99 | 70.6 | 2.31 |
| | 6 ×4 ×7/16 | 13.61 | 164.4 | 3.48 | 54.8 | 3.00 | 70.6 | 2.28 |

| Size of Web Plate. | Size of Angles. | Total Area. | Axis A–A. Moment of Inertia. | Axis A–A. Radius of Gyration. | Axis A–A. Section Modulus. | Axis A–A. Centroid. | Axis B–B. Moment of Inertia. | Axis B–B. Radius of Gyration. |
|---|---|---|---|---|---|---|---|---|
| | | A | $I_A$ | $r_A$ | $S_A$ | e | $I_B$ | $r_B$ |
| In. | In. | In.² | In.⁴ | In. | In.³ | In. | In.⁴ | In. |
| 12×½ | 4 ×3 ×½ | 12.50 | 168.6 | 3.67 | 49.1 | 3.43 | 26.4 | 1.46 |
| | 5 ×3 ×3/8 | 11.72 | 166.4 | 3.76 | 46.8 | 3.55 | 36.5 | 1.77 |
| | 5 ×3 ×½ | 13.50 | 178.2 | 3.63 | 55.9 | 3.19 | 48.9 | 1.90 |
| | 5 ×3½×½ | 14.00 | 178.2 | 3.56 | 55.7 | 3.20 | 49.7 | 1.88 |
| | 6 ×3½×3/8 | 12.84 | 174.1 | 3.69 | 52.0 | 3.35 | 61.6 | 2.19 |
| | 6 ×3½×½ | 15.00 | 186.3 | 3.52 | 62.1 | 3.00 | 82.0 | 2.34 |
| | 6 ×4 ×½ | 15.50 | 186.3 | 3.52 | 61.5 | 3.03 | 82.5 | 2.31 |
| 14×3/8 | 4 ×3 ×3/8 | 10.21 | 196.5 | 4.39 | 47.8 | 4.11 | 18.6 | 1.35 |
| | 5 ×3 ×3/8 | 10.97 | 207.4 | 4.34 | 54.1 | 3.83 | 35.1 | 1.79 |
| | 5 ×3½×3/8 | 11.35 | 207.5 | 4.28 | 54.4 | 3.81 | 35.3 | 1.76 |
| | 6 ×3½×3/8 | 12.09 | 216.6 | 4.23 | 60.5 | 3.59 | 59.6 | 2.22 |
| | 6 ×4 ×3/8 | 12.47 | 216.7 | 4.16 | 60.5 | 3.59 | 59.6 | 2.19 |
| 14×½ | 4 ×3 ×½ | 13.50 | 258.2 | 4.37 | 62.2 | 4.16 | 26.4 | 1.40 |
| | 5 ×3 ×½ | 14.50 | 273.3 | 4.34 | 70.1 | 3.89 | 48.9 | 1.84 |
| | 5 ×3½×½ | 15.00 | 273.5 | 4.27 | 70.8 | 3.87 | 49.2 | 1.81 |
| | 6 ×3½×3/8 | 13.84 | 265.7 | 4.38 | 65.3 | 4.07 | 61.6 | 2.11 |
| | 6 ×3½×½ | 16.00 | 285.3 | 4.22 | 78.3 | 3.64 | 82.0 | 2.26 |
| | 6 ×4 ×½ | 16.50 | 285.0 | 4.16 | 78.1 | 3.65 | 82.5 | 2.24 |
| 16×3/8 | 5 ×3½×3/8 | 12.10 | 299.6 | 4.98 | 66.4 | 4.52 | 35.3 | 1.70 |
| | 6 ×3½×3/8 | 12.84 | 312.6 | 4.94 | 73.3 | 4.27 | 59.7 | 2.16 |
| | 6 ×3½×½ | 15.00 | 334.7 | 4.72 | 88.1 | 3.80 | 80.0 | 2.30 |
| 16×½ | 6 ×3½×3/8 | 14.84 | 382.5 | 5.09 | 79.5 | 4.81 | 61.6 | 2.04 |
| | 6 ×3½×7/16 | 15.94 | 399.0 | 5.03 | 87.5 | 4.55 | 71.9 | 2.13 |
| | 6 ×3½×½ | 17.00 | 412.4 | 4.92 | 95.5 | 4.32 | 82.0 | 2.18 |
| | 6 ×4 ×½ | 17.50 | 412.1 | 4.85 | 95.6 | 4.31 | 82.5 | 2.17 |

## TABLE 13.

### Properties of Top Chord Sections.

Properties of Two Angles and One Cover Plate. Angles Turned Out.



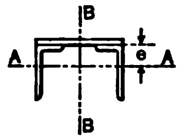

Short Legs Against Plate, and Turned Out. Edges of Angles Flush with Edges of Plate.

| Series 1 and 2. | Series 1. | | | | | | | | Series 2. | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Axis A-A. | | | | Axis B-B. | | | | Axis A-A. | | | | Axis B-B. | |
| Size of Plate. | Size of Angles. | Total Area. | Moment of Inertia. | Radius of Gyration. | Section Modulus, Upper Fiber. | Centroid. | Moment of Inertia. | Radius of Gyration. | Size of Angles. | Total Area. | Moment of Inertia. | Radius of Gyration. | Section Modulus, Upper Fiber. | Centroid. | Moment of Inertia. | Radius of Gyration. |
| | | A | $I_A$ | $r_A$ | $S_A$ | e | $I_B$ | $r_B$ | | A | $I_A$ | $r_A$ | $S_A$ | e | $I_B$ | $r_B$ |
| In. | In. | In.² | In.⁴ | In. | In.³ | In. | In.⁴ | In. | In. | In.² | In.⁴ | In. | In.³ | In. | In.⁴ | In. |
| 10x¼ | 3x2½x¼ | 5.12 | 3.7 | .86 | 5.8 | .40 | 48.5 | 3.08 | 3x2½x⅜ | 6.34 | 5.1 | .90 | 6.6 | .53 | 62.5 | 3.14 |
| | 4x3 x¼ | 5.88 | 8.2 | 1.18 | 9.0 | .66 | 49.0 | 2.89 | 4x3 x⅜ | 7.46 | 11.2 | 1.23 | 10.6 | .81 | 63.0 | 2.91 |
| 10x5/16 | 3x2½x¼ | 5.74 | 4.0 | .84 | 6.3 | .33 | 53.7 | 3.05 | 3x2½x⅜ | 6.96 | 5.6 | .90 | 7.3 | .46 | 67.7 | 3.12 |
| | 4x3 x¼ | 6.50 | 8.7 | 1.16 | 10.0 | .57 | 54.2 | 2.89 | 4x3 x⅜ | 8.08 | 11.9 | 1.22 | 11.5 | .73 | 68.2 | 2.90 |
| 12x¼ | 3x2½x¼ | 5.62 | 3.9 | .83 | 6.4 | .36 | 82.8 | 3.84 | 3x2½x⅜ | 6.84 | 5.3 | .89 | 7.4 | .48 | 106.2 | 3.94 |
| | 4x3 x¼ | 6.38 | 8.5 | 1.16 | 10.2 | .60 | 86.1 | 3.67 | 4x3 x⅜ | 7.96 | 11.6 | 1.21 | 11.7 | .75 | 110.7 | 3.73 |
| | 5x3½x5/16 | 8.12 | 18.8 | 1.52 | 15.5 | .96 | 98.6 | 3.48 | 5x3½x7/16 | 10.06 | 24.3 | 1.56 | 17.9 | 1.11 | 124.0 | 3.51 |
| 12x5/16 | 3x2½x¼ | 6.37 | 4.1 | .80 | 6.9 | .28 | 91.8 | 3.79 | 3x2½x⅜ | 7.59 | 5.7 | .87 | 8.0 | .41 | 115.2 | 3.89 |
| | 4x3 x¼ | 7.13 | 9.1 | 1.13 | 11.1 | .51 | 95.1 | 3.65 | 4x3x ⅜ | 8.71 | 12.4 | 1.19 | 12.7 | .66 | 119.7 | 3.71 |
| | 5x3½x5/16 | 8.87 | 19.8 | 1.49 | 17.1 | .85 | 107.6 | 3.48 | 5x3½x7/16 | 10.81 | 25.6 | 1.54 | 19.4 | 1.01 | 133.0 | 3.51 |
| 12x⅜ | 3x2½x¼ | 7.12 | 4.4 | .79 | 7.5 | .22 | 100.8 | 3.76 | 3x2½x⅜ | 8.34 | 6.1 | .86 | 8.6 | .34 | 124.2 | 3.86 |
| | 4x3 x¼ | 7.88 | 9.5 | 1.10 | 11.9 | .43 | 104.1 | 3.64 | 4x3 x⅜ | 9.46 | 13.0 | 1.18 | 13.8 | .58 | 128.7 | 3.69 |
| | 5x3½x5/16 | 9.62 | 20.8 | 1.47 | 18.4 | .76 | 116.6 | 3.48 | 5x3½x7/16 | 11.56 | 26.9 | 1.53 | 20.7 | .92 | 142.0 | 3.50 |
| 14x¼ | 3x2½x¼ | 6.12 | 4.0 | .81 | 7.0 | .32 | 128.4 | 4.58 | 3x2½x⅜ | 7.34 | 5.5 | .87 | 8.1 | .44 | 163.5 | 4.72 |
| | 4x3x ¼ | 6.88 | 8.8 | 1.13 | 11.0 | .55 | 135.9 | 4.45 | 4x3 x⅜ | 8.46 | 12.0 | 1.19 | 12.7 | .70 | 174.3 | 4.54 |
| | 5x3½x5/16 | 8.62 | 19.3 | 1.50 | 17.0 | .89 | 159.1 | 4.30 | 5x3½x7/16 | 10.56 | 25.0 | 1.54 | 19.2 | 1.05 | 199.8 | 4.35 |
| | 6x4 x⅜ | 10.72 | 37.1 | 1.86 | 24.4 | 1.27 | 179.1 | 4.09 | 6x4 x½ | 13.00 | 46.2 | 1.88 | 27.7 | 1.42 | 220.9 | 4.12 |
| 14x5/16 | 3x2½x¼ | 6.99 | 4.2 | .78 | 7.7 | .24 | 142.7 | 4.52 | 3x2½x⅜ | 8.21 | 5.9 | .85 | 8.7 | .37 | 177.7 | 4.65 |
| | 4x3 x¼ | 7.75 | 9.3 | 1.11 | 12.3 | .45 | 150.2 | 4.40 | 4x3 x⅜ | 9.33 | 12.8 | 1.17 | 13.9 | .61 | 188.6 | 4.49 |
| | 5x3½x5/16 | 9.49 | 20.4 | 1.47 | 18.7 | .78 | 173.4 | 4.27 | 5x3½x7/16 | 11.43 | 26.4 | 1.52 | 20.9 | .95 | 214.1 | 4.33 |
| | 6x4 x⅜ | 11.59 | 39.0 | 1.83 | 26.7 | 1.15 | 193.4 | 4.08 | 6x4 x½ | 13.87 | 48.6 | 1.87 | 30.0 | 1.31 | 235.1 | 4.11 |
| 14x⅜ | 3x2½x¼ | 7.87 | 4.5 | .76 | 8.2 | .18 | 157.0 | 4.47 | 3x2½x⅜ | 9.09 | 6.3 | .83 | 9.4 | .30 | 192.0 | 4.59 |
| | 4x3 x¼ | 8.63 | 10.2 | 1.07 | 13.1 | .37 | 164.5 | 4.37 | 4x3 x⅜ | 10.21 | 13.5 | 1.15 | 14.8 | .53 | 202.9 | 4.46 |
| | 5x3½x5/16 | 10.37 | 21.4 | 1.44 | 20.2 | .69 | 187.7 | 4.25 | 5x3½x7/16 | 12.31 | 27.6 | 1.50 | 22.4 | .86 | 228.4 | 4.31 |
| | 6x4 x⅜ | 12.47 | 40.8 | 1.81 | 28.7 | 1.04 | 207.7 | 4.08 | 6x4 x½ | 14.75 | 50.8 | 1.85 | 32.0 | 1.22 | 249.5 | 4.11 |
| 16x¼ | 4x3 x¼ | 7.38 | 9.0 | 1.10 | 12.0 | .50 | 199.5 | 5.20 | 4x3 x⅜ | 8.96 | 12.3 | 1.18 | 13.8 | .65 | 254.8 | 5.33 |
| | 5x3½x5/16 | 9.12 | 19.8 | 1.47 | 18.2 | .84 | 236.8 | 5.09 | 5x3½x7/16 | 11.06 | 25.7 | 1.52 | 20.6 | 1.00 | 296.9 | 5.18 |
| | 6x4 x⅜ | 11.22 | 38.0 | 1.84 | 26.2 | 1.20 | 271.3 | 4.91 | 6x4 x½ | 13.50 | 47.4 | 1.87 | 27.4 | 1.36 | 334.4 | 4.98 |
| 16x5/16 | 4x3 x¼ | 8.38 | 9.5 | 1.07 | 13.2 | .41 | 220.9 | 5.13 | 4x3 x⅜ | 9.96 | 13.1 | 1.15 | 15.1 | .56 | 276.2 | 5.27 |
| | 5x3½x5/16 | 10.12 | 20.9 | 1.44 | 20.1 | .73 | 258.1 | 5.05 | 5x3½x7/16 | 12.06 | 27.1 | 1.50 | 22.6 | .89 | 318.2 | 5.14 |
| | 6x4 x⅜ | 12.22 | 42.0 | 1.81 | 28.8 | 1.08 | 292.7 | 4.90 | 6x4 x½ | 14.50 | 49.9 | 1.85 | 32.0 | 1.25 | 355.7 | 4.95 |
| 16x⅜ | 5x3½x5/16 | 11.12 | 21.9 | 1.40 | 21.8 | .63 | 279.4 | 5.02 | 5x3½x7/16 | 13.06 | 28.5 | 1.48 | 24.4 | .80 | 339.6 | 5.10 |
| | 6x4 x⅜ | 13.22 | 41.9 | 1.78 | 31.0 | .98 | 314.0 | 4.87 | 6x4 x½ | 15.50 | 52.2 | 1.83 | 34.3 | 1.15 | 377.0 | 4.93 |
| | 8x6 x7/16 | 17.86 | 106.0 | 2.44 | 54.7 | 1.56 | 307.8 | 4.15 | 8x6 x9/16 | 21.12 | 129.6 | 2.48 | 61.4 | 1.74 | 361.3 | 4.13 |

## TABLE 14.

### PROPERTIES OF TOP CHORD SECTIONS.

**Properties of Two Angles and One Cover Plate. Angles Turned In.**



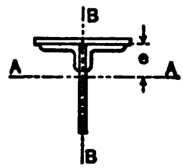

**Short Legs Against Plate, and Turned In. Backs of Angles Flush with Edges of Plate.**

| Series 1 and 2. | Series 1. | | | | | | | | Series 2. | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Axis A–A. | | | | Axis B–B. | | | | Axis A–A. | | | | Axis B–B. | |
| Size of Plate. | Size of Angles. | Total Area. | Moment of Inertia. | Radius of Gyration. | Section Modulus, Upper Fiber. | Centroid. | Moment of Inertia. | Radius of Gyration. | Size of Angles. | Total Area. | Moment of Inertia. | Radius of Gyration. | Section Modulus, Upper Fiber. | Centroid. | Moment of Inertia. | Radius of Gyration. |
| | | A | $I_A$ | $r_A$ | $S_A$ | e | $I_B$ | $r_B$ | | A | $I_A$ | $r_A$ | $S_A$ | e | $I_B$ | $r_B$ |
| In. | In. | In.² | In.⁴ | In. | In.³ | In. | In.⁴ | In. | In. | In.² | In.⁴ | In. | In.³ | In. | In.⁴ | In. |
| 8x¼ | 3x2½x¼ | 4.62 | 3.6 | 0.88 | 5.1 | .46 | 41.4 | 2.99 | 3x2½x⅜ | 5.84 | 4.9 | .91 | 5.8 | .59 | 54.3 | 3.05 |
| " | 4x3 x¼ | 5.38 | 7.9 | 1.21 | 8.1 | .73 | 49.4 | 3.03 | 4x3 x⅜ | 6.96 | 10.8 | 1.25 | 9.6 | .88 | 66.0 | 3.08 |
| 8x5/16 | 3x2½x¼ | 5.12 | 3.9 | 0.87 | 5.6 | .39 | 44.0 | 2.93 | 3x2½x⅜ | 6.34 | 5.3 | .91 | 6.4 | .52 | 57.0 | 3.00 |
| " | 4x3 x¼ | 5.88 | 8.4 | 1.20 | 8.7 | .65 | 52.1 | 2.98 | 4x3 x⅜ | 7.46 | 11.4 | 1.24 | 10.3 | .80 | 68.6 | 3.03 |
| 10x¼ | 3x2½x¼ | 5.12 | 3.8 | 0.86 | 5.8 | .41 | 71.7 | 3.74 | 3x2½x⅜ | 6.34 | 5.2 | .90 | 6.6 | .53 | 93.6 | 3.84 |
| " | 4x3 x¼ | 5.88 | 8.4 | 1.19 | 9.2 | .66 | 85.0 | 3.80 | 4x3 x⅜ | 7.46 | 11.3 | 1.23 | 10.6 | .81 | 113.0 | 3.89 |
| " | 5x3½x5/16 | 7.62 | 18.1 | 1.54 | 14.1 | 1.03 | 114.9 | 3.88 | 5x3½x7/16 | 9.56 | 23.5 | 1.57 | 16.5 | 1.17 | 147.9 | 3.93 |
| " | 6x4 x⅜ | 9.72 | 34.9 | 1.89 | 21.0 | 1.41 | 149.6 | 3.92 | 6x4 x½ | 12.00 | 43.7 | 1.91 | 24.3 | 1.55 | 186.1 | 3.94 |
| 10x5/16 | 3x2½x¼ | 5.74 | 4.1 | 0.83 | 6.2 | .33 | 76.9 | 3.66 | 3x2½x⅜ | 6.96 | 5.6 | .90 | 7.3 | .46 | 98.8 | 3.76 |
| " | 4x3 x¼ | 6.50 | 8.8 | 1.16 | 10.0 | .57 | 90.2 | 3.72 | 4x3 x⅜ | 8.08 | 12.0 | 1.22 | 11.5 | .73 | 118.2 | 3.82 |
| " | 5x3½x5/16 | 8.24 | 19.2 | 1.53 | 15.5 | .93 | 120.1 | 3.82 | 5x3½x7/16 | 10.18 | 24.7 | 1.56 | 17.8 | 1.08 | 153.2 | 3.88 |
| " | 6x4 x⅜ | 10.34 | 36.7 | 1.88 | 22.6 | 1.31 | 154.9 | 3.87 | 6x4 x½ | 12.62 | 45.6 | 1.90 | 25.8 | 1.46 | 191.3 | 3.89 |
| 10x⅜ | 3x2½x¼ | 6.37 | 4.2 | 0.81 | 6.6 | .26 | 82.1 | 3.59 | 3x2½x⅜ | 7.59 | 5.9 | .88 | 7.7 | .39 | 104.0 | 3.70 |
| " | 4x3 x¼ | 7.13 | 9.3 | 1.14 | 10.6 | .49 | 95.4 | 3.66 | 4x3 x⅜ | 8.71 | 12.6 | 1.20 | 12.2 | .66 | 123.4 | 3.76 |
| " | 5x3½x5/16 | 8.87 | 22.0 | 1.50 | 16.5 | .84 | 125.4 | 3.76 | 5x3½x7/16 | 10.81 | 25.9 | 1.54 | 18.8 | 1.00 | 158.4 | 3.83 |
| " | 6x4 x⅜ | 10.97 | 38.2 | 1.87 | 24.0 | 1.21 | 160.0 | 3.82 | 6x4 x½ | 13.25 | 47.5 | 1.89 | 27.3 | 1.37 | 196.5 | 3.85 |
| 12x¼ | 4x3 x¼ | 6.38 | 8.6 | 1.16 | 10.2 | .60 | 132.3 | 4.55 | 4x3 x⅜ | 7.96 | 11.7 | 1.21 | 11.7 | .75 | 175.0 | 4.69 |
| " | 5x3½x5/16 | 8.12 | 18.8 | 1.52 | 15.5 | .96 | 177.8 | 4.68 | 5x3½x7/16 | 10.06 | 24.3 | 1.56 | 17.9 | 1.11 | 228.4 | 4.76 |
| " | 6x4 x⅜ | 10.22 | 36.0 | 1.88 | 22.8 | 1.33 | 230.6 | 4.76 | 6x4 x½ | 12.50 | 45.0 | 1.90 | 26.0 | 1.48 | 287.0 | 4.79 |
| 12x5/16 | 4x3 x¼ | 7.13 | 9.1 | 1.13 | 11.1 | .51 | 141.3 | 4.45 | 4x3 x⅜ | 8.71 | 12.4 | 1.19 | 12.7 | .66 | 184.0 | 4.60 |
| " | 5x3½x5/16 | 8.87 | 19.8 | 1.49 | 17.1 | .85 | 186.8 | 4.59 | 5x3½x7/16 | 10.81 | 25.6 | 1.54 | 19.4 | 1.01 | 237.6 | 4.69 |
| " | 6x4 x⅜ | 10.97 | 37.9 | 1.86 | 24.8 | 1.22 | 239.6 | 4.67 | 6x4 x½ | 13.25 | 47.2 | 1.89 | 27.9 | 1.38 | 296.0 | 4.73 |
| 12x⅜ | 4x3 x¼ | 7.88 | 9.5 | 1.10 | 11.9 | .43 | 150.3 | 4.37 | 4x3 x⅜ | 9.46 | 13.1 | 1.18 | 13.8 | .58 | 193.0 | 4.52 |
| " | 5x3½x5/16 | 9.62 | 20.8 | 1.47 | 18.4 | .76 | 195.8 | 4.51 | 5x3½x7/16 | 11.56 | 26.9 | 1.53 | 20.7 | .92 | 246.6 | 4.62 |
| " | 6x4 x⅜ | 11.72 | 39.6 | 1.84 | 26.4 | 1.12 | 248.6 | 4.61 | 6x4 x½ | 14.00 | 49.2 | 1.87 | 29.6 | 1.29 | 305.0 | 4.67 |
| 14x¼ | 4x3 x¼ | 6.88 | 8.8 | 1.13 | 11.0 | .55 | 192.4 | 5.29 | 4x3 x⅜ | 8.46 | 12.0 | 1.19 | 12.7 | .70 | 252.9 | 5.47 |
| " | 5x3½x5/16 | 8.62 | 19.3 | 1.50 | 17.0 | .89 | 257.0 | 5.46 | 5x3½x7/16 | 10.56 | 25.0 | 1.54 | 19.2 | 1.05 | 328.9 | 5.58 |
| " | 6x4 x⅜ | 10.72 | 37.1 | 1.86 | 24.4 | 1.27 | 332.2 | 5.56 | 6x4 x½ | 13.00 | 46.2 | 1.88 | 27.7 | 1.42 | 412.9 | 5.63 |
| 14x5/16 | 4x3 x¼ | 7.75 | 9.3 | 1.11 | 12.3 | .45 | 206.7 | 5.16 | 4x3 x⅜ | 9.33 | 12.8 | 1.17 | 13.9 | .61 | 267.2 | 5.34 |
| " | 5x3½x5/16 | 9.49 | 20.4 | 1.47 | 18.7 | .78 | 271.3 | 5.34 | 5x3½x7/16 | 11.43 | 26.4 | 1.52 | 20.9 | .95 | 343.1 | 5.48 |
| " | 6x4 x⅜ | 11.59 | 39.0 | 1.83 | 26.7 | 1.15 | 346.4 | 5.46 | 6x4 x½ | 13.87 | 48.6 | 1.87 | 30.0 | 1.31 | 427.2 | 5.54 |
| 14x⅜ | 4x3 x¼ | 8.63 | 9.9 | 1.07 | 13.1 | .37 | 221.0 | 5.06 | 4x3 x⅜ | 10.21 | 13.5 | 1.15 | 14.8 | .53 | 281.5 | 5.25 |
| " | 5x3½x5/16 | 10.37 | 21.4 | 1.44 | 20.2 | .69 | 285.5 | 5.24 | 5x3½x7/16 | 12.31 | 27.6 | 1.50 | 22.4 | .86 | 357.4 | 5.39 |
| " | 6x4 x⅜ | 12.47 | 40.8 | 1.81 | 28.9 | 1.04 | 360.6 | 5.38 | 6x4 x½ | 14.75 | 50.8 | 1.85 | 32.0 | 1.22 | 441.4 | 5.47 |
| " | 8x6 x7/16 | 17.11 | 103.7 | 2.46 | 51.6 | 1.64 | 489.7 | 5.35 | 8x6 x9/16 | 20.37 | 126.7 | 2.49 | 58.1 | 1.81 | 591.2 | 5.40 |

# TABLE 15.

## Properties of Top Chord Sections.

**Properties of Two Angles, One Web Plate and One Cover Plate.**



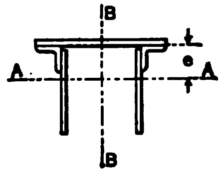

**Long Legs Turned Out. Top of Web Plate ¼" Below Backs of Angles.**

| Series 1 and 2. | | Series 1. | | | | | | | | Series 2. | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Axis A–A. | | | | Axis B–B. | | | | Axis A–A. | | | | Axis B–B. | |
| Size of Web Plate. | Size of Angles. | Size of Top Plate. | Total Area. | Moment of Inertia. | Radius of Gyration. | Section Modulus, Upper Fiber. | Centroid. | Moment of Inertia. | Radius of Gyration. | Size of Top Plate. | Total Area. | Moment of Inertia. | Radius of Gyration. | Section Modulus, Upper Fiber. | Centroid. | Moment of Inertia. | Radius of Gyration. |
| | | | A | $I_A$ | $r_A$ | $S_A$ | e | $I_B$ | $r_B$ | | A | $I_A$ | $r_A$ | $S_A$ | e | $I_B$ | $r_B$ |
| In. | In. | In. | In.² | In.⁴ | In. | In.³ | In. | In.⁴ | In. | In. | In.² | In.⁴ | In. | In.³ | In. | In.⁴ | In. |
| 6x¼ | 2 x2 x¼ | 6x¼ | 4.88 | 14.8 | 1.74 | 10.3 | 1.19 | 6.1 | 1.12 | 6x⅜ | 5.63 | 16.2 | 1.70 | 11.8 | .99 | 8.4 | 1.22 |
| 8x¼ | 2 x2 x¼ | 6x¼ | 5.38 | 31.6 | 2.42 | 15.8 | 1.75 | 6.1 | 1.07 | 6x⅜ | 6.13 | 34.5 | 2.37 | 18.4 | 1.50 | 8.4 | 1.17 |
| | 2½x2½x¼ | 6x¼ | 5.88 | 32.3 | 2.34 | 16.5 | 1.71 | 7.6 | 1.14 | 6x⅜ | 6.63 | 35.0 | 2.30 | 18.9 | 1.48 | 9.9 | 1.22 |
| | " 5/16 | 6x¼ | 6.44 | 32.9 | 2.26 | 17.5 | 1.63 | 8.4 | 1.14 | 6x⅜ | 7.19 | 35.5 | 2.22 | 19.7 | 1.43 | 10.7 | 1.22 |
| | 3 x2½x¼ | 8x¼ | 6.62 | 34.4 | 2.28 | 19.5 | 1.51 | 15.8 | 1.55 | 8x⅜ | 7.62 | 37.1 | 2.21 | 22.5 | 1.27 | 21.2 | 1.67 |
| | " 5/16 | 8x¼ | 7.24 | 35.3 | 2.21 | 20.8 | 1.45 | 17.1 | 1.54 | 8x⅜ | 8.24 | 37.7 | 2.14 | 23.5 | 1.23 | 22.5 | 1.65 |
| 8x5/16 | 2½x2½x¼ | 6x¼ | 6.38 | 38.0 | 2.44 | 17.6 | 1.91 | 7.6 | 1.10 | 6x⅜ | 7.13 | 41.3 | 2.41 | 20.2 | 1.67 | 10.0 | 1.18 |
| | " 5/16 | 6x¼ | 6.94 | 38.9 | 2.37 | 18.8 | 1.82 | 8.4 | 1.10 | 6x⅜ | 7.69 | 41.9 | 2.33 | 21.1 | 1.61 | 10.8 | 1.18 |
| | 3 x2½x¼ | 8x¼ | 7.12 | 40.5 | 2.38 | 20.8 | 1.70 | 16.0 | 1.49 | 8x⅜ | 8.12 | 44.0 | 2.33 | 24.1 | 1.45 | 21.3 | 1.62 |
| | " 5/16 | 8x¼ | 7.74 | 41.4 | 2.31 | 22.0 | 1.63 | 17.3 | 1.49 | 8x⅜ | 8.74 | 44.7 | 2.26 | 25.2 | 1.40 | 22.7 | 1.61 |
| 8x⅜ | 3 x2½x¼ | 8x¼ | 7.62 | 46.3 | 2.46 | 21.8 | 1.87 | 16.2 | 1.46 | 8x⅜ | 8.62 | 48.5 | 2.37 | 23.0 | 1.73 | 21.5 | 1.58 |
| | " 5/16 | 8x¼ | 8.24 | 47.3 | 2.39 | 23.3 | 1.78 | 17.6 | 1.46 | 8x⅜ | 9.24 | 49.4 | 2.31 | 24.1 | 1.67 | 22.9 | 1.57 |
| | 4 x3 x5/16 | 10x⅜ | 10.93 | 54.9 | 2.24 | 31.1 | 1.40 | 46.8 | 2.07 | 10x½ | 12.18 | 58.6 | 2.19 | 34.3 | 1.21 | 57.2 | 2.17 |
| | " ⅜ | 10x⅜ | 11.71 | 55.5 | 2.18 | 32.1 | 1.36 | 49.9 | 2.06 | 10x½ | 12.96 | 59.2 | 2.14 | 35.1 | 1.19 | 60.3 | 2.16 |
| 10x¼ | 2½x2½x¼ | 6x¼ | 6.38 | 58.1 | 3.06 | 23.0 | 2.30 | 7.6 | 1.09 | 6x⅜ | 7.13 | 63.4 | 2.96 | 26.1 | 2.02 | 9.9 | 1.18 |
| | " 5/16 | 6x¼ | 6.94 | 60.0 | 2.94 | 24.7 | 2.18 | 8.4 | 1.10 | 6x⅜ | 7.69 | 64.4 | 2.89 | 27.9 | 1.93 | 10.7 | 1.18 |
| | 3 x2½x¼ | 8x¼ | 7.12 | 62.4 | 2.96 | 27.1 | 2.05 | 15.8 | 1.49 | 8x⅜ | 8.12 | 67.2 | 2.88 | 31.5 | 1.76 | 21.2 | 1.62 |
| | " 5/16 | 8x¼ | 7.74 | 64.3 | 2.88 | 29.0 | 1.94 | 17.1 | 1.49 | 8x⅜ | 8.74 | 68.3 | 2.81 | 33.0 | 1.70 | 22.5 | 1.60 |
| | 4 x3 x5/16 | 10x⅜ | 10.43 | 72.4 | 2.63 | 38.6 | 1.50 | 46.1 | 2.10 | 10x½ | 11.68 | 76.5 | 2.56 | 42.7 | 1.29 | 56.5 | 2.20 |
| | " ⅜ | 10x⅜ | 11.21 | 73.0 | 2.55 | 41.1 | 1.45 | 49.9 | 2.09 | 10x½ | 12.46 | 77.0 | 2.49 | 43.7 | 1.26 | 59.4 | 2.18 |
| 10x5/16 | 3 x2½x¼ | 8x¼ | 7.75 | 73.5 | 3.08 | 28.7 | 2.31 | 16.0 | 1.43 | 8x⅜ | 8.75 | 79.6 | 2.99 | 32.9 | 2.01 | 21.3 | 1.56 |
| | " 5/16 | 8x¼ | 8.37 | 75.3 | 3.00 | 30.8 | 2.20 | 17.4 | 1.43 | 8x⅜ | 9.37 | 80.9 | 2.95 | 35.3 | 1.94 | 22.7 | 1.56 |
| | 4 x3 x5/16 | 10x⅜ | 11.06 | 85.8 | 2.79 | 41.1 | 1.71 | 46.4 | 2.06 | 10x½ | 12.31 | 91.0 | 2.75 | 46.7 | 1.49 | 56.9 | 2.15 |
| | " ⅜ | 10x⅜ | 11.84 | 87.0 | 2.71 | 42.8 | 1.66 | 49.4 | 2.05 | 10x½ | 13.09 | 91.8 | 2.69 | 48.5 | 1.45 | 59.9 | 2.14 |
| | 5 x3½x5/16 | 12x⅜ | 12.75 | 90.5 | 2.66 | 46.8 | 1.56 | 82.8 | 2.56 | 12x½ | 14.25 | 95.8 | 2.59 | 51.8 | 1.35 | 100.8 | 2.66 |
| | " ⅜ | 12x⅜ | 13.73 | 91.9 | 2.59 | 49.0 | 1.50 | 88.6 | 2.55 | 12x½ | 15.23 | 96.9 | 2.52 | 53.0 | 1.33 | 106.6 | 2.64 |
| 10x⅜ | 3 x2½x¼ | 8x¼ | 8.37 | 83.7 | 3.16 | 30.1 | 2.53 | 16.2 | 1.38 | 8x⅜ | 9.37 | 90.8 | 3.11 | 34.9 | 2.23 | 21.5 | 1.51 |
| | " 5/16 | 8x¼ | 8.99 | 85.8 | 3.10 | 32.4 | 2.42 | 17.6 | 1.40 | 8x⅜ | 9.99 | 92.5 | 3.05 | 36.9 | 2.15 | 22.9 | 1.51 |
| | 4 x3 x5/16 | 10x⅜ | 11.68 | 98.4 | 2.92 | 43.2 | 1.90 | 46.8 | 2.00 | 10x½ | 12.93 | 104.6 | 2.81 | 47.3 | 1.67 | 57.2 | 2.10 |
| | " ⅜ | 10x⅜ | 12.46 | 99.7 | 2.83 | 45.2 | 1.84 | 49.9 | 2.00 | 10x½ | 13.71 | 105.4 | 2.77 | 49.5 | 1.63 | 60.3 | 2.10 |
| | 5 x3½x5/16 | 12x⅜ | 13.37 | 103.7 | 2.78 | 49.4 | 1.73 | 83.4 | 2.50 | 12x½ | 14.87 | 110.0 | 2.72 | 54.7 | 1.51 | 101.4 | 2.61 |
| | " ⅜ | 12x⅜ | 14.35 | 105.3 | 2.71 | 51.4 | 1.68 | 89.3 | 2.50 | 12x½ | 15.85 | 111.5 | 2.65 | 56.4 | 1.48 | 107.3 | 2.60 |

# TABLE 16.

## PROPERTIES OF TOP CHORD SECTIONS.

Properties of Two Angles, Two Web Plates and One Cover Plate.



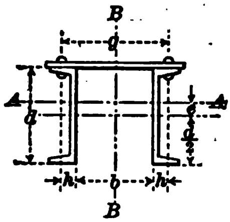

Angle Legs Turned Out. Edges of Angles Flush with Edges of Top Plate. Web Plates ½" Below Backs of Angles.

| Series 1 and 2. | | Series 1. | | | | | | | | Series 2. | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | Axis A–A. | | | | Axis B–B. | | | | Axis A–A. | | | | Axis B–B. | |
| Size of Web Plates. | Size of Angles. | Size of Top Plate. | Total Area | Moment of Inertia. | Radius of Gyration. | Section Modulus, Upper Fiber. | Centroid. | Moment of Inertia. | Radius of Gyration. | Size of Top Plate. | Total Area. | Moment of Inertia. | Radius of Gyration. | Section Modulus, Upper Fiber. | Centroid. | Moment of Inertia. | Radius of Gyration. |
| | | | A | $I_A$ | $r_A$ | $S_A$ | e | $I_B$ | $r_B$ | | A | $I_A$ | $r_A$ | $S_A$ | e | $I_B$ | $r_B$ |
| In. | In. | In. | In.² | In.⁴ | In. | In.³ | In. | In.⁴ | In. | In. | In.² | In.⁴ | In. | In.³ | In. | In.⁴ | In. |
| 8x¼ | 2½x2½x¼ | 10x¼ | 8.88 | 58 | 2.56 | 25.1 | 2.07 | 69.5 | 2.80 | 10x⅜ | 10.13 | 64 | 2.52 | 29.9 | 1.78 | 79.9 | 2.81 |
| " ⅜ | " | " | 10.88 | 76 | 2.64 | 27.9 | 2.47 | 79.9 | 2.7[illegible] | " | 12.13 | 86 | 2.66 | 33.3 | 2.19 | 90.3 | 2.73 |
| " ¼ | 2½x2½x⅜ | " | 9.96 | 60 | 2.45 | 27.4 | 1.94 | 82.2 | 2.87 | " | 11.21 | 66 | 2.43 | 31.8 | 1.69 | 92.6 | 2.87 |
| " ⅜ | " | " | 11.96 | 80 | 2.58 | 30.8 | 2.33 | 91.6 | 2.77 | " | 13.21 | 88 | 2.57 | 35.6 | 2.08 | 102.1 | 2.78 |
| 8x¼ | 2½x2½x¼ | 12x¼ | 9.38 | 60 | 2.53 | 27.5 | 1.95 | 125.4 | 3.66 | 12x⅜ | 10.88 | 67 | 2.48 | 33.3 | 1.64 | 143.4 | 3.63 |
| " ⅜ | " | " | 11.38 | 80 | 2.65 | 30.7 | 2.36 | 145.7 | 3.57 | " | 12.88 | 89 | 2.63 | 36.8 | 2.05 | 163.7 | 3.56 |
| " ¼ | 2½x2½x⅜ | " | 10.46 | 62 | 2.44 | 29.7 | 1.84 | 146.4 | 3.74 | " | 11.96 | 68 | 2.39 | 35.0 | 1.57 | 164.4 | 3.71 |
| " ⅜ | " | " | 12.46 | 83 | 2.58 | 33.3 | 2.23 | 166.7 | 3.65 | " | 13.96 | 91 | 2.56 | 39.2 | 1.95 | 184.7 | 3.64 |
| 10x¼ | 2½x2½x¼ | 12x¼ | 10.38 | 109 | 3.24 | 37.5 | 2.66 | 136.8 | 3.63 | 12x⅜ | 11.88 | 120 | 3.18 | 45.3 | 2.28 | 154.8 | 3.61 |
| " ⅜ | " | " | 12.88 | 143 | 3.33 | 41.9 | 3.16 | 162.2 | 3.55 | " | 14.38 | 159 | 3.33 | 50.1 | 2.80 | 180.2 | 3.54 |
| " ¼ | 2½x2½x⅜ | " | 11.46 | 113 | 3.14 | 41.2 | 2.49 | 157.8 | 3.71 | " | 12.96 | 123 | 3.08 | 48.4 | 2.17 | 175.8 | 3.68 |
| " ⅜ | " | " | 13.96 | 149 | 3.27 | 46.1 | 2.98 | 183.2 | 3.62 | " | 15.46 | 164 | 3.25 | 53.9 | 2.66 | 201.2 | 3.61 |
| 10x¼ | 2½x2½x¼ | 14x¼ | 10.88 | 113 | 3.22 | 40.5 | 2.53 | 219.1 | 4.47 | 14x⅜ | 12.63 | 125 | 3.14 | 49.6 | 2.14 | 247.8 | 4.43 |
| " ⅜ | " | " | 13.38 | 149 | 3.34 | 45.3 | 3.04 | 262.9 | 4.43 | " | 15.13 | 166 | 3.31 | 54.8 | 2.65 | 291.6 | 4.39 |
| " ¼ | 2½x2½x⅜ | " | 11.96 | 116 | 3.12 | 43.9 | 2.38 | 250.6 | 4.58 | " | 13.71 | 127 | 3.04 | 52.7 | 2.04 | 279.2 | 4.51 |
| " ⅜ | " | " | 14.46 | 154 | 3.26 | 46.3 | 2.88 | 294.4 | 4.51 | " | 16.21 | 170 | 3.23 | 58.3 | 2.53 | 323.0 | 4.46 |
| 12x⅜ | 3x3x¼ | 14x¼ | 15.38 | 244 | 3.98 | 60.4 | 3.79 | 258.1 | 4.10 | 14x⅜ | 17.13 | 270 | 3.97 | 72.2 | 3.37 | 286.7 | 4.09 |
| " ½ | " | " | 18.38 | 295 | 4.01 | 66.5 | 4.19 | 296.1 | 4.01 | " | 20.13 | 328 | 4.03 | 78.5 | 3.80 | 324.8 | 4.02 |
| " ⅜ | 3x3x⅜ | " | 16.72 | 254 | 3.90 | 66.7 | 3.56 | 292.6 | 4.18 | " | 18.47 | 279 | 3.88 | 77.9 | 3.20 | 321.2 | 4.17 |
| " ½ | " | " | 19.72 | 309 | 3.96 | 73.2 | 3.97 | 330.7 | 4.09 | " | 21.47 | 339 | 3.97 | 84.7 | 3.62 | 359.3 | 4.09 |
| 12x⅜ | 3x3x¼ | 16x⅜ | 17.88 | 280 | 3.96 | 77.7 | 3.22 | 437.3 | 4.94 | 16x½ | 19.88 | 304 | 3.91 | 90.6 | 2.85 | 480.0 | 4.91 |
| " ½ | " | " | 20.88 | 339 | 4.03 | 84.3 | 3.65 | 499.9 | 4.89 | " | 22.88 | 370 | 4.02 | 97.4 | 3.30 | 542.6 | 4.87 |
| " ⅜ | 3x3x⅜ | " | 19.22 | 286 | 3.86 | 83.5 | 3.06 | 486.3 | 5.03 | " | 21.22 | 309 | 3.82 | 95.7 | 2.73 | 529.0 | 4.99 |
| " ½ | " | " | 22.22 | 348 | 3.96 | 90.1 | 3.50 | 548.9 | 4.97 | " | 24.22 | 377 | 3.95 | 102.8 | 3.17 | 591.6 | 4.94 |
| 14x⅜ | 3x3x⅜ | 16x⅜ | 20.72 | 431 | 4.56 | 103.2 | 3.80 | 521.1 | 5.01 | 16x½ | 22.72 | 464 | 4.52 | 118.1 | 3.43 | 563.8 | 4.98 |
| " ½ | " | " | 24.22 | 524 | 4.65 | 112.1 | 4.30 | 594.0 | 4.95 | " | 26.22 | 565 | 4.64 | 127.3 | 3.94 | 636.7 | 4.93 |
| " ⅜ | 3x3x½ | " | 22.00 | 441 | 4.48 | 109.9 | 3.64 | 569.0 | 5.08 | " | 24.00 | 472 | 4.44 | 124.1 | 3.31 | 621.7 | 5.09 |
| " ½ | " | " | 25.50 | 537 | 4.59 | 119.1 | 4.13 | 641.9 | 5.02 | " | 27.50 | 577 | 4.58 | 133.8 | 3.81 | 684.6 | 4.99 |
| 14x⅜ | 3x3x⅜ | 18x⅜ | 21.47 | 443 | 4.54 | 109.7 | 3.66 | 740.9 | 5.87 | 18x½ | 23.72 | 477 | 4.49 | 126.5 | 3.28 | 801.6 | 5.81 |
| " ½ | " | " | 24.97 | 539 | 4.64 | 118.6 | 4.17 | 849.1 | 5.83 | " | 27.22 | 582 | 4.63 | 136.0 | 3.79 | 909.8 | 5.78 |
| " ⅜ | 3x3x½ | " | 22.75 | 452 | 4.46 | 116.1 | 3.52 | 805.6 | 5.95 | " | 25.00 | 484 | 4.40 | 132.2 | 3.16 | 866.3 | 5.89 |
| " ½ | " | " | 26.25 | 551 | 4.58 | 125.6 | 4.02 | 913.8 | 5.90 | " | 28.50 | 593 | 4.56 | 142.4 | 3.66 | 974.5 | 5.85 |

## TABLE 17.

### PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



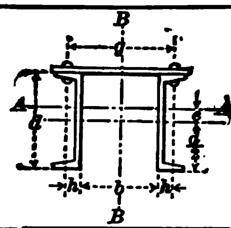

Two Channels and One Plate.

| Section Number. | Channels. | | Cover Plate. | B to B Channels. | Total Area. | Eccentricity. | Moments of Inertia. | | Radii of Gyration. | | Gages. | | Web of Channels. | Max. Rivet. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | Depth. | Weight. | | | | | Axis A–A. | Axis B–B. | Axis A–A. | Axis B–B. | Plate. | Channels. | | |
| | | | | b | | e | $I_A$ | $I_B$ | $r_A$ | $r_B$ | g | h | | |
| | In. | Lb. | In. | In. | In.² | In. | In.⁴ | In.⁴ | In. | In. | In. | In. | In. | In. |
| 1 | 5 | 6.50 | 8×1/4 | 3 7/8 | 5.90 | 0.89 | 23.9 | 34.7 | 2.01 | 2.42 | 6 | 1 1/16 | .19 | 1/2 |
| 2 | | | 8×5/16 | “ | 6.40 | 1.04 | 25.6 | 37.3 | 2.00 | 2.41 | “ | “ | “ | “ |
| 3 | | | 10×1/4 | 5 7/8 | 6.40 | 1.03 | 25.3 | 67.6 | 1.99 | 3.25 | 8 | “ | “ | “ |
| 4 | | | 10×5/16 | “ | 7.03 | 1.18 | 27.1 | 72.8 | 1.96 | 3.22 | “ | “ | “ | “ |
| 5 | | | 12×1/4 | 7 7/8 | 6.90 | 1.14 | 26.5 | 113.5 | 1.96 | 4.05 | 10 | “ | “ | “ |
| 6 | | | 12×5/16 | “ | 7.65 | 1.30 | 28.3 | 122.5 | 1.92 | 4.00 | “ | “ | “ | “ |
| 7 | 5 | 9.00 | 8×1/4 | 3 5/8 | 7.30 | 0.72 | 27.8 | 39.9 | 1.95 | 2.34 | 6 | 1 3/16 | .33 | 1/2 |
| 8 | | | 8×5/16 | “ | 7.80 | 0.84 | 29.7 | 42.5 | 1.95 | 2.33 | “ | “ | “ | “ |
| 9 | | | 10×1/4 | 5 5/8 | 7.80 | 0.84 | 29.5 | 79.7 | 1.95 | 3.20 | 8 | “ | “ | “ |
| 10 | | | 10×5/16 | “ | 8.43 | 0.99 | 31.7 | 84.9 | 1.94 | 3.17 | “ | “ | “ | “ |
| 11 | | | 12×1/4 | 7 5/8 | 8.30 | 0.95 | 31.0 | 135.1 | 1.93 | 4.04 | 10 | “ | “ | “ |
| 12 | | | 12×5/16 | “ | 9.05 | 1.10 | 33.3 | 144.1 | 1.92 | 3.99 | “ | “ | “ | “ |
| 13 | 6 | 8.00 | 10×1/4 | 5 1/2 | 7.26 | 1.08 | 42.0 | 73.1 | 2.41 | 3.17 | 7 3/4 | 1 1/8 | .20 | 3/8 |
| 14 | | | 10×5/16 | “ | 7.89 | 1.25 | 44.8 | 78.3 | 2.38 | 3.15 | “ | “ | “ | “ |
| 15 | | | 12×1/4 | 7 1/2 | 7.76 | 1.21 | 44.0 | 124.0 | 2.38 | 4.00 | 9 3/4 | “ | “ | “ |
| 16 | | | 12×5/16 | “ | 8.51 | 1.39 | 46.9 | 133.0 | 2.35 | 3.95 | “ | “ | “ | “ |
| 17 | | | 14×5/16 | 9 1/2 | 9.14 | 1.51 | 48.7 | 204.9 | 2.31 | 4.74 | 11 3/4 | “ | “ | “ |
| 18 | | | 14×3/8 | “ | 10.01 | 1.67 | 51.3 | 219.2 | 2.26 | 4.67 | “ | “ | “ | “ |
| 19 | 6 | 10.50 | 10×1/4 | 5 1/4 | 8.68 | 0.90 | 47.6 | 83.1 | 2.34 | 3.09 | 7 3/4 | 1 1/4 | .32 | 5/8 |
| 20 | | | 10×5/16 | “ | 9.31 | 1.06 | 50.9 | 88.3 | 2.34 | 3.08 | “ | “ | “ | “ |
| 21 | | | 12×1/4 | 7 1/4 | 9.18 | 1.02 | 50.0 | 143.0 | 2.33 | 3.95 | 9 3/4 | “ | “ | “ |
| 22 | | | 12×5/16 | “ | 9.93 | 1.19 | 53.5 | 152.0 | 2.32 | 3.91 | “ | “ | “ | “ |
| 23 | | | 14×5/16 | 9 1/4 | 10.56 | 1.31 | 55.8 | 235.7 | 2.30 | 4.73 | 11 3/4 | “ | “ | “ |
| 24 | | | 14×3/8 | “ | 11.43 | 1.47 | 58.9 | 250.3 | 2.27 | 4.68 | “ | “ | “ | “ |
| 25 | 7 | 9.75 | 10×1/4 | 5 1/4 | 8.20 | 1.11 | 65.1 | 80.1 | 2.82 | 3.13 | 7 3/4 | 1 1/4 | .21 | 5/8 |
| 26 | | | 10×5/16 | “ | 8.83 | 1.30 | 69.2 | 85.3 | 2.80 | 3.11 | “ | “ | “ | “ |
| 27 | | | 12×1/4 | 7 1/4 | 8.70 | 1.25 | 68.0 | 137.1 | 2.80 | 3.97 | 9 3/4 | “ | “ | “ |
| 28 | | | 12×5/16 | “ | 9.45 | 1.45 | 72.5 | 146.1 | 2.77 | 3.93 | “ | “ | “ | “ |
| 29 | | | 14×5/16 | 9 1/4 | 10.08 | 1.59 | 75.3 | 225.8 | 2.73 | 4.73 | 11 3/4 | “ | “ | “ |
| 30 | | | 14×3/8 | “ | 10.95 | 1.77 | 79.3 | 240.1 | 2.69 | 4.68 | “ | “ | “ | “ |
| 31 | 7 | 12.25 | 10×1/4 | 5 1/8 | 9.70 | 0.93 | 72.8 | 92.1 | 2.74 | 3.08 | 7 3/4 | 1 5/16 | .32 | 5/8 |
| 32 | | | 10×5/16 | “ | 10.33 | 1.11 | 77.5 | 97.3 | 2.74 | 3.07 | “ | “ | “ | “ |
| 33 | | | 12×1/4 | 7 1/8 | 10.20 | 1.07 | 76.2 | 159.1 | 2.73 | 3.95 | 9 3/4 | “ | “ | “ |
| 34 | | | 12×5/16 | “ | 10.95 | 1.25 | 81.4 | 168.1 | 2.73 | 3.92 | “ | “ | “ | “ |
| 35 | | | 14×5/16 | 9 1/8 | 11.58 | 1.38 | 84.8 | 260.7 | 2.71 | 4.74 | 11 3/4 | “ | “ | “ |
| 36 | | | 14×3/8 | “ | 12.45 | 1.55 | 89.8 | 275.0 | 2.69 | 4.70 | “ | “ | “ | “ |

## TABLE 17.—*Continued.*

### Properties of Top Chord Sections.

Properties of Top Chord Sections.

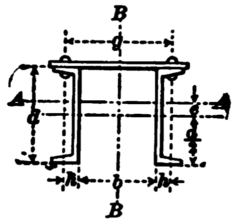



Two Channels and One Plate.

| Section Number. | Channels. Depth. | Channels. Weight. | Cover Plate. | B to B Channels. b | Gross Area. | Eccentricity. e | Moments of Inertia. Axis A–A. $I_A$ | Moments of Inertia. Axis B–B. $I_B$ | Radii of Gyration. Axis A–A. $r_A$ | Radii of Gyration. Axis B–B. $r_B$ | Gages. Plate. g | Gages. Channel. h | Web of Channel. | Max. Rivet. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | In. | Lb. | In. | In. | In.$^2$ | In. | In.$^4$ | In.$^4$ | In. | In. | In. | In. | In. | In. |
| 37 | 8 | 11.25 | 12×1/4 | 7 | 9.70 | 1.28 | 99.9 | 150.2 | 3.21 | 3.93 | 9 1/2 | 1 1/4 | .22 | 3/4 |
| 38 | | | 12×5/16 | " | 10.45 | 1.49 | 106.2 | 159.3 | 3.19 | 3.90 | " | " | " | " |
| 39 | | | 14×5/16 | 9 | 11.08 | 1.64 | 110.4 | 247.2 | 3.16 | 4.72 | 11 1/2 | " | " | " |
| 40 | | | 14×3/8 | " | 11.95 | 1.84 | 116.3 | 261.4 | 3.12 | 4.67 | " | " | " | " |
| 41 | | | 16×3/8 | 11 | 12.70 | 1.98 | 120.2 | 378.5 | 3.08 | 5.46 | 13 1/2 | " | " | " |
| 42 | | | 16×7/16 | " | 13.70 | 2.16 | 125.4 | 400.0 | 3.03 | 5.40 | " | " | " | " |
| 43 | 8 | 13.75 | 12×1/4 | 6 7/8 | 11.08 | 1.12 | 109.2 | 168.3 | 3.14 | 3.90 | 9 1/2 | 1 5/16 | .31 | 3/4 |
| 44 | | | 12×5/16 | " | 11.83 | 1.32 | 116.3 | 177.3 | 3.13 | 3.87 | " | " | " | " |
| 45 | | | 14×5/16 | 8 7/8 | 12.46 | 1.46 | 121.0 | 276.6 | 3.12 | 4.71 | 11 1/2 | " | " | " |
| 46 | | | 14×3/8 | " | 13.33 | 1.65 | 127.8 | 290.9 | 3.10 | 4.67 | " | " | " | " |
| 47 | | | 16×3/8 | 10 7/8 | 14.08 | 1.78 | 132.5 | 421.9 | 3.07 | 5.48 | 13 1/2 | " | " | " |
| 48 | | | 16×7/16 | " | 15.08 | 1.96 | 138.7 | 443.2 | 3.03 | 5.42 | " | " | " | " |
| 49 | 9 | 13.25 | 12×1/4 | 6 3/4 | 10.78 | 1.29 | 140.9 | 162.9 | 3.62 | 3.89 | 9 1/2 | 1 3/8 | .23 | 3/4 |
| 50 | | | 12×5/16 | " | 11.53 | 1.51 | 149.5 | 171.9 | 3.60 | 3.86 | " | " | " | " |
| 51 | | | 14×5/16 | 8 3/4 | 12.16 | 1.68 | 155.3 | 268.2 | 3.57 | 4.70 | 11 1/2 | " | " | " |
| 52 | | | 14×3/8 | " | 13.03 | 1.89 | 163.5 | 282.4 | 3.54 | 4.66 | " | " | " | " |
| 53 | | | 16×3/8 | 10 3/4 | 13.78 | 2.04 | 169.1 | 409.9 | 3.50 | 5.45 | 13 1/2 | " | " | " |
| 54 | | | 16×7/16 | " | 14.78 | 2.23 | 176.8 | 431.3 | 3.46 | 5.40 | " | " | " | " |
| 55 | 9 | 15.00 | 12×1/4 | 6 5/8 | 11.82 | 1.17 | 149.7 | 174.1 | 3.56 | 3.84 | 9 1/2 | 1 7/16 | .29 | 3/4 |
| 56 | | | 12×5/16 | " | 12.57 | 1.39 | 158.8 | 183.1 | 3.55 | 3.82 | " | " | " | " |
| 57 | | | 14×5/16 | 8 5/8 | 13.20 | 1.54 | 165.2 | 287.4 | 3.54 | 4.67 | 11 1/2 | " | " | " |
| 58 | | | 14×3/8 | " | 14.07 | 1.75 | 174.2 | 301.7 | 3.52 | 4.63 | " | " | " | " |
| 59 | | | 16×3/8 | 10 5/8 | 14.82 | 1.90 | 180.3 | 439.4 | 3.49 | 5.44 | 13 1/2 | " | " | " |
| 60 | | | 16×7/16 | " | 15.82 | 2.09 | 188.6 | 460.7 | 3.45 | 5.40 | " | " | " | " |
| 61 | 10 | 15.00 | 14×5/16 | 8 1/2 | 13.30 | 1.70 | 211.7 | 289.4 | 3.99 | 4.67 | 11 1/2 | 1 1/2 | .24 | 3/4 |
| 62 | | | 14×3/8 | " | 14.17 | 1.92 | 222.8 | 303.6 | 3.97 | 4.63 | " | " | " | " |
| 63 | | | 16×3/8 | 10 1/2 | 14.92 | 2.09 | 230.4 | 441.9 | 3.93 | 5.44 | 13 1/2 | " | " | " |
| 64 | | | 16×7/16 | " | 15.92 | 2.30 | 240.6 | 463.9 | 3.89 | 5.39 | " | " | " | " |
| 65 | | | 18×7/16 | 12 1/2 | 16.80 | 2.45 | 247.7 | 641.2 | 3.84 | 6.18 | 15 1/2 | " | " | " |
| 66 | | | 18×1/2 | " | 17.92 | 2.64 | 257.1 | 671.6 | 3.79 | 6.12 | " | " | " | " |
| 67 | 10 | 20.00 | 14×5/16 | 8 1/4 | 16.14 | 1.40 | 242.1 | 341.2 | 3.88 | 4.60 | 11 1/2 | 1 5/8 | .38 | 3/4 |
| 68 | | | 14×3/8 | " | 17.01 | 1.60 | 255.2 | 355.0 | 3.87 | 4.57 | " | " | " | " |
| 69 | | | 16×3/8 | 10 1/4 | 17.76 | 1.75 | 264.4 | 520.4 | 3.86 | 5.41 | 13 1/2 | " | " | " |
| 70 | | | 16×7/16 | " | 18.76 | 1.95 | 276.9 | 542.0 | 3.84 | 5.37 | " | " | " | " |
| 71 | | | 18×7/16 | 12 1/4 | 19.64 | 2.09 | 286.9 | 752.3 | 3.82 | 6.19 | 15 1/2 | " | " | " |
| 72 | | | 18×1/2 | " | 20.76 | 2.28 | 297.8 | 782.7 | 3.79 | 6.14 | " | " | " | " |
| 73 | 10 | 25.00 | 14×5/16 | 7 7/8 | 19.08 | 1.18 | 271.8 | 383.9 | 3.77 | 4.48 | 11 1/2 | 1 11/16 | .53 | 3/4 |
| 74 | | | 14×3/8 | " | 19.95 | 1.37 | 286.2 | 398.2 | 3.79 | 4.47 | " | " | " | " |
| 75 | | | 16×3/8 | 9 7/8 | 20.70 | 1.50 | 296.8 | 588.8 | 3.79 | 5.33 | 13 1/2 | " | " | " |
| 76 | | | 16×7/16 | " | 21.70 | 1.62 | 313.6 | 610.1 | 3.80 | 5.30 | " | " | " | " |
| 77 | | | 18×7/16 | 11 7/8 | 22.58 | 1.73 | 325.2 | 851.4 | 3.79 | 6.14 | 15 1/2 | " | " | " |
| 78 | | | 18×1/2 | " | 23.70 | 1.99 | 336.0 | 881.8 | 3.77 | 6.10 | " | " | " | " |

## TABLE 17.—*Continued.*
### PROPERTIES OF TOP CHORD SECTIONS.

Properties of Top Chord Sections.



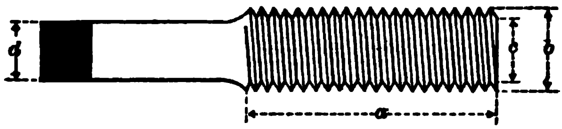

Two Channels and One Plate.

| Section Number. | Channels. Depth. | Channels. Weight. | Cover Plate. | B to B Channels. b | Total Area. | Eccentricity. e | Moments of Inertia. Axis A–A. $I_A$ | Moments of Inertia. Axis B–B. $I_B$ | Radii of Gyration. Axis A–A. $r_A$ | Radii of Gyration. Axis B–B. $r_B$ | Gages. Plate. g | Gages. Channels. h | Web of Channels. | Max. Rivet. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | In. | Lb. | In. | In. | In.² | In. | In.⁴ | In.⁴ | In. | In. | In. | In. | In. | In. |
| 79 | 12 | 20.50 | 16×3/8 | 9 3/8 | 18.06 | 2.06 | 409.8 | 485.8 | 4.76 | 5.19 | 13 | 1 13/16 | .28 | 7/8 |
| 80 | | | 16×7/16 | " | 19.06 | 2.28 | 427.6 | 507.1 | 4.74 | 5.16 | " | " | " | " |
| 81 | | | 18×3/8 | 11 3/8 | 18.91 | 2.21 | 422.4 | 682.1 | 4.73 | 6.00 | 15 | " | " | " |
| 82 | | | 18×7/16 | " | 19.94 | 2.46 | 440.6 | 712.4 | 4.70 | 5.98 | " | " | " | " |
| 83 | | | 20×7/16 | 13 3/8 | 20.81 | 2.62 | 452.5 | 957.5 | 4.66 | 6.78 | 17 | " | " | " |
| 84 | | | 20×1/2 | " | 22.06 | 2.83 | 469.8 | 999.1 | 4.61 | 6.73 | " | " | " | " |
| 85 | 12 | 25.00 | 16×3/8 | 9 1/4 | 20.70 | 1.79 | 451.4 | 550.0 | 4.67 | 5.16 | 13 | 1 7/8 | .39 | 7/8 |
| 86 | | | 16×7/16 | " | 21.70 | 2.01 | 471.5 | 571.3 | 4.66 | 5.13 | " | " | " | " |
| 87 | | | 18×3/8 | 11 1/4 | 21.45 | 1.95 | 465.1 | 774.9 | 4.66 | 6.01 | 15 | " | " | " |
| 88 | | | 18×7/16 | " | 22.58 | 2.17 | 486.5 | 805.2 | 4.64 | 5.98 | " | " | " | " |
| 89 | | | 20×7/16 | 13 1/4 | 23.45 | 2.32 | 500.3 | 1084.7 | 4.62 | 6.80 | 17 | " | " | " |
| 90 | | | 20×1/2 | " | 24.70 | 2.53 | 520.5 | 1126.3 | 4.59 | 6.75 | " | " | " | " |
| 91 | 12 | 30.00 | 16×3/8 | 9 | 23.64 | 1.57 | 494.9 | 611.4 | 4.58 | 5.08 | 13 | 2 | .51 | 7/8 |
| 92 | | | 16×7/16 | " | 24.64 | 1.77 | 517.3 | 632.7 | 4.58 | 5.06 | " | " | " | " |
| 93 | | | 18×3/8 | 11 | 24.39 | 1.71 | 510.1 | 865.7 | 4.57 | 5.96 | 15 | " | " | " |
| 94 | | | 18×7/16 | " | 25.52 | 1.92 | 534.1 | 896.0 | 4.58 | 5.93 | " | " | " | " |
| 95 | | | 20×7/16 | 13 | 26.39 | 2.06 | 549.8 | 1211.1 | 4.56 | 6.78 | 17 | " | " | " |
| 96 | | | 20×1/2 | " | 27.64 | 2.34 | 567.6 | 1252.7 | 4.53 | 6.73 | " | " | " | " |
| 97 | 15 | 33.00 | 18×3/8 | 10 5/8 | 26.55 | 1.96 | 922.8 | 936.7 | 5.90 | 5.94 | 15 | 2 3/16 | .40 | 7/8 |
| 98 | | | 18×7/16 | " | 27.68 | 2.20 | 961.0 | 967.0 | 5.89 | 5.91 | " | " | " | " |
| 99 | | | 20×7/16 | 12 5/8 | 28.55 | 2.36 | 986.7 | 1307.1 | 5.88 | 6.76 | 17 | " | " | " |
| 100 | | | 20×1/2 | " | 29.80 | 2.60 | 1024.5 | 1348.7 | 5.86 | 6.72 | " | " | " | " |
| 101 | | | 22×1/2 | 14 5/8 | 30.80 | 2.77 | 1050.2 | 1761.1 | 5.84 | 7.56 | 19 | " | " | " |
| 102 | | | 22×9/16 | " | 32.18 | 3.00 | 1085.5 | 1816.5 | 5.81 | 7.50 | " | " | " | " |
| 103 | 15 | 35.00 | 18×3/8 | 10 5/8 | 27.33 | 1.90 | 940.5 | 965.7 | 5.87 | 5.95 | 15 | 2 3/16 | .43 | 7/8 |
| 104 | | | 18×7/16 | " | 28.46 | 2.14 | 979.7 | 996.0 | 5.87 | 5.92 | " | " | " | " |
| 105 | | | 20×7/16 | 12 5/8 | 29.33 | 2.30 | 1005.6 | 1346.7 | 5.86 | 6.78 | 17 | " | " | " |
| 106 | | | 20×1/2 | " | 30.58 | 2.53 | 1044.4 | 1388.3 | 5.84 | 6.74 | " | " | " | " |
| 107 | | | 22×1/2 | 14 5/8 | 31.58 | 2.70 | 1070.8 | 1811.7 | 5.82 | 7.58 | 19 | " | " | " |
| 108 | | | 22×9/16 | " | 32.96 | 2.92 | 1107.9 | 1867.1 | 5.79 | 7.52 | " | " | " | " |
| 109 | 15 | 40.00 | 18×3/8 | 10 3/8 | 30.27 | 1.71 | 1005.1 | 1039.3 | 5.76 | 5.86 | 15 | 2 5/16 | .52 | 7/8 |
| 110 | | | 18×7/16 | " | 31.40 | 1.94 | 1047.0 | 1069.6 | 5.77 | 5.84 | " | " | " | " |
| 111 | | | 20×7/16 | 12 3/8 | 32.27 | 2.09 | 1074.8 | 1453.5 | 5.77 | 6.71 | 17 | " | " | " |
| 112 | | | 20×1/2 | " | 33.52 | 2.31 | 1116.7 | 1495.1 | 5.77 | 6.68 | " | " | " | " |
| 113 | | | 22×1/2 | 14 3/8 | 34.52 | 2.47 | 1145.4 | 1956.5 | 5.76 | 7.52 | 19 | " | " | " |
| 114 | | | 22×9/16 | " | 35.90 | 2.68 | 1186.2 | 2011.9 | 5.75 | 7.48 | " | " | " | " |
| 115 | 15 | 45.00 | 18×3/8 | 10 1/4 | 33.23 | 1.56 | 1068.2 | 1127.9 | 5.67 | 5.82 | 15 | 2 3/8 | .62 | 7/8 |
| 116 | | | 18×7/16 | " | 34.36 | 1.77 | 1112.0 | 1158.2 | 5.69 | 5.81 | " | " | " | " |
| 117 | | | 20×7/16 | 12 1/4 | 35.23 | 1.92 | 1141.9 | 1577.3 | 5.69 | 6.69 | 17 | " | " | " |
| 118 | | | 20×1/2 | " | 36.48 | 2.12 | 1186.4 | 1618.9 | 5.70 | 6.66 | " | " | " | " |
| 119 | | | 22×1/2 | 14 1/4 | 37.48 | 2.28 | 1217.2 | 2120.7 | 5.70 | 7.52 | 19 | " | " | " |
| 120 | | | 22×9/16 | " | 38.86 | 2.48 | 1260.6 | 2176.1 | 5.70 | 7.48 | " | " | " | " |

## TABLE 18.

### Upset Screw Ends for Square Bars.

**AMERICAN BRIDGE COMPANY STANDARD.**




Pitch and Shape of Thread A. B. Co. Standard.

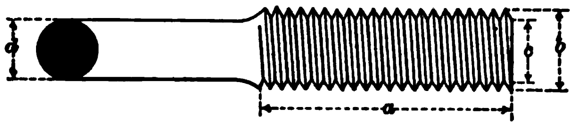
| BAR. | | | UPSET. | | | | | |
|---|---|---|---|---|---|---|---|---|
| Side of Square d, Inches. | Area, Sq. Inches. | Weight per Foot, Lbs. | Diameter b, Inches. | Length a, Inches. | Additional Length for Upset +10%, Inches. | Diameter at Root of Thread c, Inches. | Area. At Root of Thread, Sq. Inches. | Area. Excess Over Area of Bar, %. |
| * 3/4 | 0.563 | 1.91 | 1 1/8 | 4 | 4 | 0.939 | 0.693 | 23.2 |
| * 7/8 | 0.766 | 2.60 | 1 1/4 | 4 | 3 1/2 | 1.064 | 0.890 | 16.2 |
| 1 | 1.000 | 3.40 | 1 1/2 | 4 | 4 | 1.283 | 1.294 | 29.4 |
| 1 1/8 | 1.266 | 4.30 | 1 5/8 | 4 | 3 1/2 | 1.389 | 1.515 | 19.7 |
| 1 1/4 | 1.563 | 5.31 | 1 7/8 | 4 1/2 | 4 1/2 | 1.615 | 2.049 | 31.1 |
| 1 3/8 | 1.891 | 6.43 | 2 | 4 1/2 | 4 | 1.711 | 2.300 | 21.7 |
| 1 1/2 | 2.250 | 7.65 | 2 1/4 | 5 | 5 | 1.961 | 3.021 | 34.3 |
| 1 5/8 | 2.641 | 8.98 | 2 3/8 | 5 | 4 1/2 | 2.086 | 3.419 | 29.5 |
| 1 3/4 | 3.063 | 10.41 | 2 1/2 | 5 1/2 | 4 1/2 | 2.175 | 3.716 | 21.3 |
| 1 7/8 | 3.516 | 11.95 | 2 3/4 | 5 1/2 | 5 | 2.425 | 4.619 | 31.4 |
| 2 | 4.000 | 13.60 | 2 7/8 | 6 | 5 | 2.550 | 5.108 | 27.7 |
| 2 1/8 | 4.516 | 15.35 | 3 | 6 | 4 1/2 | 2.629 | 5.428 | 20.2 |
| 2 1/4 | 5.063 | 17.21 | 3 1/4 | 6 1/2 | 5 1/2 | 2.879 | 6.509 | 28.6 |
| 2 3/8 | 5.641 | 19.18 | 3 1/2 | 7 | 6 1/2 | 3.100 | 7.549 | 33.8 |
| 2 1/2 | 6.250 | 21.25 | 3 3/4 | 7 | 7 | 3.317 | 8.641 | 38.3 |
| 2 5/8 | 6.891 | 23.43 | 3 3/4 | 7 | 5 1/2 | 3.317 | 8.641 | 25.4 |
| 2 3/4 | 7.563 | 25.71 | 4 | 7 1/2 | 6 1/2 | 3.567 | 9.993 | 32.1 |
| 2 7/8 | 8.266 | 28.10 | 4 1/4 | 8 | 7 1/2 | 3.798 | 11.330 | 37.1 |
| 3 | 9.000 | 30.60 | 4 1/4 | 8 | 6 | 3.798 | 11.330 | 25.9 |
| 3 1/8 | 9.766 | 33.20 | 4 1/2 | 8 1/2 | 7 | 4.028 | 12.741 | 30.5 |
| 3 1/4 | 10.563 | 35.91 | 4 3/4 | 8 1/2 | 7 1/2 | 4.255 | 14.221 | 34.6 |

Upsets marked * are special.

## TABLE 19.

### Upset Screw Ends for Round Bars.

AMERICAN BRIDGE COMPANY STANDARD.




Pitch and Shape of Thread A. B. Co. Standard.

| BAR. | | | UPSET. | | | | | |
|---|---|---|---|---|---|---|---|---|
| Diameter d, Inches. | Area, Sq. Inches. | Weight per Foot, Lb. | Diameter b, Inches. | Length a, Inches. | Additional Length for Upset +10 %, Inches. | Diameter at Root of Thread c, Inches. | Area. At Root of Thread, Sq. Inches. | Area. Excess Over Area of Bar, %. |
| * 3/4 | 0.442 | 1.50 | 1 | 4 | 4 | 0.838 | 0.551 | 24.7 |
| * 7/8 | 0.601 | 2.04 | 1 1/4 | 4 | 5 | 1.064 | 0.890 | 48.0 |
| 1 | 0.785 | 2.67 | 1 3/8 | 4 | 4 | 1.158 | 1.054 | 34.2 |
| 1 1/8 | 0.994 | 3.38 | 1 1/2 | 4 | 4 | 1.283 | 1.294 | 30.2 |
| 1 1/4 | 1.227 | 4.17 | 1 5/8 | 4 | 4 | 1.389 | 1.515 | 23.5 |
| 1 3/8 | 1.485 | 5.05 | 1 3/4 | 4 | 4 | 1.490 | 1.744 | 17.5 |
| 1 1/2 | 1.767 | 6.01 | 2 | 4 1/2 | 4 1/2 | 1.711 | 2.300 | 30.2 |
| 1 5/8 | 2.074 | 7.05 | 2 1/8 | 4 1/2 | 4 | 1.836 | 2.649 | 27.7 |
| 1 3/4 | 2.405 | 8.18 | 2 1/4 | 5 | 4 | 1.961 | 3.021 | 25.6 |
| 1 7/8 | 2.761 | 9.39 | 2 3/8 | 5 | 4 | 2.086 | 3.419 | 23.8 |
| 2 | 3.142 | 10.68 | 2 1/2 | 5 1/2 | 4 | 2.175 | 3.716 | 18.3 |
| 2 1/8 | 3.547 | 12.06 | 2 5/8 | 5 1/2 | 3 1/2 | 2.300 | 4.156 | 17.2 |
| 2 1/4 | 3.976 | 13.52 | 2 7/8 | 6 | 4 1/2 | 2.550 | 5.108 | 28.4 |
| 2 3/8 | 4.430 | 15.06 | 3 | 6 | 4 1/2 | 2.629 | 5.428 | 22.5 |
| 2 1/2 | 4.909 | 16.69 | 3 1/4 | 6 1/2 | 5 1/2 | 2.879 | 6.509 | 32.6 |
| 2 5/8 | 5.412 | 18.40 | 3 1/4 | 6 1/2 | 4 1/2 | 2.879 | 6.509 | 20.3 |
| 2 3/4 | 5.940 | 20.19 | 3 1/2 | 7 | 5 1/2 | 3.100 | 7.549 | 27.1 |
| 2 7/8 | 6.492 | 22.07 | 3 3/4 | 7 | 6 | 3.317 | 8.641 | 33.1 |
| 3 | 7.069 | 24.03 | 3 3/4 | 7 | 5 | 3.317 | 8.641 | 22.2 |
| 3 1/8 | 7.670 | 26.08 | 4 | 7 1/2 | 6 | 3.567 | 9.993 | 30.3 |
| 3 1/4 | 8.296 | 28.21 | 4 | 7 1/2 | 5 | 3.567 | 9.993 | 20.5 |
| 3 3/8 | 8.946 | 30.42 | 4 1/4 | 8 | 5 1/2 | 3.798 | 11.330 | 26.6 |
| 3 1/2 | 9.621 | 32.71 | 4 1/4 | 8 | 5 | 3.798 | 11.330 | 17.8 |
| 3 5/8 | 10.321 | 35.09 | 4 1/2 | 8 1/2 | 5 1/2 | 4.028 | 12.741 | 23.4 |
| 3 3/4 | 11.045 | 37.55 | 4 3/4 | 8 1/2 | 6 | 4.255 | 14.221 | 28.8 |
| 3 7/8 | 11.793 | 40.10 | 4 3/4 | 8 1/2 | 5 1/2 | 4.255 | 14.221 | 20.6 |

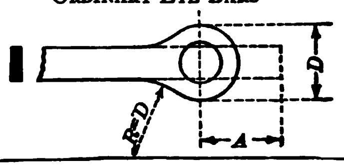
Upsets marked * are special.

## TABLE 20.

### Standard Eye Bars

### American Bridge Company Standards

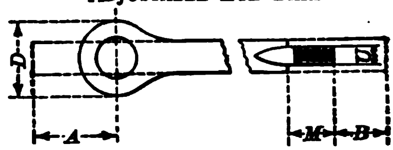
#### Ordinary Eye Bars




| Bar: Width, In. | Bar: Thickness, Max., In. | Bar: Thickness, Min., In. | Head: Dia. D, In. | Max. Pin: Dia., In. | Max. Pin: Excess Head Over Bar, %. | Add. Material A: For Ordering Bar, Ft. & In. | Add. Material A: For Figuring Weight of Bar, Ft. & In. |
|---|---|---|---|---|---|---|---|
| 2 | 1 | ½ | 4½ | 1¾ | ...... | 1– 0 | 0– 7 |
| | | | 5½ | 2¾ | 37.5 | 1– 4 | 0–11 |
| | | | * 6½ | 3¾ | ...... | 1– 9 | 1– 4 |
| 2½ | 1 | ⅝ | 6 | 2½ | ...... | 1– 3 | 0–10 |
| | | | 7 | 3½ | 40.0 | 1– 7 | 1– 2 |
| | | | * 8 | 4½ | ...... | 2– 0 | 1– 7 |
| 3 | 1½ | ⅝ | 7½ | 3¼ | ...... | 1– 6 | 1– 1 |
| | | | 8½ | 4¼ | 41.7 | 1–11 | 1– 5 |
| | | | * 9½ | 5¼ | ...... | 2– 4 | 1–10 |
| 4 | 1¾ | ¾ | 10 | 4½ | ...... | 1–11 | 1– 6 |
| | | ⅞ | 11 | 5½ | 37.5 | 2– 3 | 1–10 |
| | | 1 | *12 | 6½ | ...... | 2– 8 | 2– 2 |
| 5 | 2 | ¾ | 12 | 5¼ | ...... | 2– 1 | 1– 8 |
| | | 1 | 13½ | 6¾ | 35.0 | 2– 8 | 2– 2 |
| | | 1 | *15 | 8¼ | ...... | 3– 3 | 2– 9 |
| 6 | 2 | ¾ | 14 | 5¾ | ...... | 2– 4 | 1–10 |
| | | 1 | 14¾ | 6½ | 37.5 | 2– 6 | 2– 1 |
| | | 1 | *16½ | 8¼ | ...... | 3– 2 | 2– 8 |
| 7 | 2 | 1 | 16½ | 7 | ...... | 2– 7 | 2– 2 |
| | | 1⅛ | 17½ | 8 | 35.7 | 2–11 | 2– 6 |
| | | 1⅛ | *18½ | 9 | ...... | 3– 4 | 2–11 |
| 8 | 2 | 1 | 18 | 7 | ...... | 2– 8 | 2– 3 |
| | | 1⅛ | 19 | 8 | 37.5 | 3– 0 | 2– 6 |
| | | 1¼ | *20 | 9 | ...... | 3– 4 | 2–11 |
| 9 | 2 | 1⅛ | 20 | 7½ | 38.9 | 2–11 | 2– 6 |
| | | 1¼ | *22 | 9½ | | 3– 7 | 3– 1 |
| 10 | 2 | 1⅛ | 22½ | 9 | ...... | 3– 5 | 2–10 |
| | | 1¼ | 24 | 10½ | 35.0 | 3 –9 | 3– 3 |
| | | 1⅜ | *25 | 11½ | ...... | 4– 1 | 3– 7 |
| 12 | 2 | 1¼ | 26½ | 10 | ...... | 3– 8 | 3– 3 |
| | | 1⅜ | 28 | 11½ | 37.5 | 4– 2 | 3– 8 |
| | | 1½ | *29½ | 13 | ...... | 4– 8 | 4– 1 |
| 14 | 2 | 1⅜ | 31 | 12 | ...... | 4– 3 | 3– 9 |
| | | 1½ | 33 | 14 | 35.7 | 4–10 | 4– 4 |
| | | 1⅝ | *34 | 15 | ...... | 5– 5 | 4– 8 |
| 16 | 2 | 1¾ | 36 | 14 | 37.5 | 4–11 | 4– 5 |
| | | 1⅞ | *37½ | 16 | 34.4 | 5– 5 | 4–10 |

Bars marked * should only be used when absolutely unavoidable.
Deduct Pin Holes when figuring weights.

#### Adjustable Eye Bars



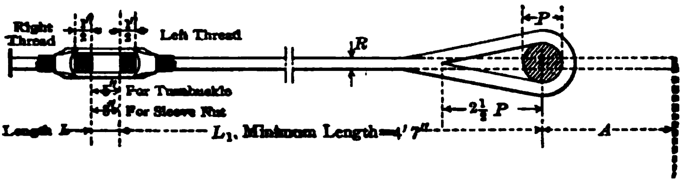

| Bar: Width, In. | Bar: Min. Thickness, In. | Screw End: Dia. U., In. | Screw End: Excess Upset Over Bar, %. | Screw End: Length M., In. | Add. Material B: For Ordering Bar, In. | Add. Material B: For Figuring Weight, In. |
|---|---|---|---|---|---|---|
| 2 | *⅝ | 1¾ | 39.6 | 4 | 12 | 8 |
| | ¾ | 1⅞ | 36.6 | 4½ | 12 | 7½ |
| | ⅞ | 2 | 31.4 | 4½ | 11 | 7½ |
| 2½ | *¾ | 2⅛ | 41.2 | 4½ | 12 | 8 |
| | ⅞ | 2¼ | 38.1 | 5 | 12 | 8 |
| | 1 | 2⅜ | 36.7 | 5 | 12 | 7½ |
| 3 | *¾ | 2¼ | 34.3 | 5 | 12 | 7½ |
| | ⅞ | 2½ | 41.6 | 5½ | 13 | 9½ |
| | 1 | 2½ | 23.9 | 5½ | 13 | 8½ |
| 4 | *¾ | 2½ | 23.9 | 5½ | 13 | 8½ |
| | ⅞ | 2¾ | 32.0 | 5½ | 11 | 7½ |
| | 1 | 3 | 35.7 | 6 | 13 | 8½ |
| | 1⅛ | 3¼ | 44.6 | 6½ | 14 | 9½ |
| 5 | *¾ | 2⅞ | 36.2 | 6 | 12 | 8 |
| | ⅞ | 3 | 24.1 | 6 | 11 | 7 |
| | 1 | 3¼ | 30.2 | 6½ | 12 | 8 |
| | 1⅛ | 3½ | 34.2 | 7 | 13 | 8½ |
| | 1¼ | 3¾ | 38.3 | 7 | 14 | 9 |
| 6 | *1 | 3½ | 25.8 | 7 | 12 | 7½ |
| | 1⅛ | 3¾ | 28.0 | 7 | 12 | 8 |
| | 1¼ | 4 | 33.2 | 7½ | 13 | 8½ |
| | 1⅜ | 4¼ | 37.3 | 8 | 14 | 9½ |
| 7 | *1⅛ | 4 | 26.9 | 7½ | 12 | 8 |
| | 1¼ | 4¼ | 29.5 | 8 | 13 | 8½ |
| | 1⅜ | 4½ | 32.4 | 8½ | 14 | 9 |
| | 1½ | 4¾ | 35.4 | 8½ | 14 | 9½ |
| 8 | *1⅛ | 4¼ | 25.9 | 8 | 12 | 8 |
| | 1¼ | 4½ | 27.4 | 8½ | 13 | 8½ |
| | 1⅜ | 4¾ | 29.3 | 8½ | 13 | 8½ |
| | 1½ | 5 | 31.4 | 9 | 14 | 9 |
| | 1⅝ | 5¼ | 35.2 | 9½ | 15 | 10 |

Bars marked * should only be used when unavoidable.
Minimum length of short end from center of pin to end of screw 6′–6″, preferably 7′–0″.
Thread on short end to be left hand.
Deduct Pin Holes when figuring weights.

# TABLE 21.

## Loop Rods.

### AMERICAN BRIDGE COMPANY STANDARD.




Pitch and Shape of Thread A. B. Co. Standard.

**Additional Length "A" in Feet and Inches for One Loop.**

$A = 4.17P + 5.89R.$

| Diam. of Pin, P. | Diameter or Side "R" of Rod in Inches. | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 3/4 | 7/8 | 1 | 1 1/8 | 1 1/4 | 1 3/8 | 1 1/2 | 1 5/8 | 1 3/4 | 1 7/8 | 2 |
| 1 1/8 | 0- 9 1/2 | 0-10 | 0-11 | 0-11 1/2 | ...... | ...... | ...... | ...... | ...... | ...... | ...... |
| 1 1/4 | 0-10 | 0-10 1/2 | 0-11 1/2 | 1- 0 | 1- 1 | ...... | ...... | ...... | ...... | ...... | ...... |
| 1 1/2 | 0-11 | 0-11 1/2 | 1- 0 1/2 | 1- 1 | 1- 2 | 1- 2 1/2 | ...... | ...... | ...... | ...... | ...... |
| 1 3/4 | 1- 0 | 1- 0 1/2 | 1- 1 1/2 | 1- 2 | 1- 3 | 1- 3 1/2 | 1- 4 1/2 | 1- 5 | 1- 6 | ...... | ...... |
| 2 | 1- 1 | 1- 1 1/2 | 1- 2 1/2 | 1- 3 | 1- 4 | 1- 4 1/2 | 1- 5 1/2 | 1- 6 | 1- 7 | 1- 7 1/2 | 1- 8 1/2 |
| 2 1/4 | 1- 2 | 1- 3 | 1- 3 1/2 | 1- 4 1/2 | 1- 5 | 1- 5 1/2 | 1- 6 1/2 | 1- 7 | 1- 8 | 1- 8 1/2 | 1- 9 1/2 |
| 2 1/2 | 1- 3 | 1- 4 | 1- 4 1/2 | 1- 5 1/2 | 1- 6 | 1- 7 | 1- 7 1/2 | 1- 8 | 1- 9 | 1- 9 1/2 | 1-10 1/2 |
| 2 3/4 | 1- 4 | 1- 5 | 1- 5 1/2 | 1- 6 1/2 | 1- 7 | 1- 8 | 1- 8 1/2 | 1- 9 1/2 | 1-10 | 1-11 | 1-11 1/2 |
| 3 | 1- 5 | 1- 6 | 1- 6 1/2 | 1- 7 1/2 | 1- 8 | 1- 9 | 1- 9 1/2 | 1-10 1/2 | 1-11 | 2- 0 | 2- 0 1/2 |
| *3 1/4 | 1- 6 | 1- 7 | 1- 7 1/2 | 1- 8 1/2 | 1- 9 | 1-10 | 1-10 1/2 | 1-11 1/2 | 2- 0 | 2- 1 | 2- 1 1/2 |
| 3 1/2 | 1- 7 1/2 | 1- 8 | 1- 8 1/2 | 1- 9 1/2 | 1-10 | 1-11 | 1-11 1/2 | 2- 0 1/2 | 2- 1 | 2- 2 | 2- 2 1/2 |
| *3 3/4 | 1- 8 1/2 | 1- 9 | 1-10 | 1-10 1/2 | 1-11 | 2- 0 | 2- 0 1/2 | 2- 1 1/2 | 2- 2 | 2- 3 | 2- 3 1/2 |
| 4 | 1- 9 1/2 | 1-10 | 1-11 | 1-11 1/2 | ?- 0 1/2 | 2- 1 | 2- 2 | 2- 2 1/2 | 2- 3 | 2- 4 | 2- 4 1/2 |
| *4 1/4 | ...... | 1-11 | 2- 0 | 2- 0 1/2 | 2- 1 1/2 | 2- 2 | 2- 3 | 2- 3 1/2 | 2- 4 1/2 | 2- 5 | 2- 6 |
| 4 1/2 | ...... | 2- 0 | 2- 1 | 2- 1 1/2 | 2- 2 1/2 | 2- 3 | 2- 4 | 2- 4 1/2 | 2- 5 1/2 | 2- 6 | 2- 7 |
| *4 3/4 | ...... | 2- 1 | 2- 2 | 2- 2 1/2 | 2- 3 1/2 | 2- 4 | 2- 5 | 2- 5 1/2 | 2- 6 1/2 | 2- 7 | 2- 8 |
| 5 | ...... | 2- 2 1/2 | 2- 3 | 2- 3 1/2 | 2- 4 1/2 | 2- 5 | 2- 6 | 2- 6 1/2 | 2- 7 1/2 | 2- 8 | 2- 9 |
| *5 1/4 | ...... | ...... | 2- 4 | 2- 5 | 2- 5 1/2 | 2- 6 | 2- 7 | 2- 7 1/2 | 2- 8 1/2 | 2- 9 | 2-10 |
| 5 1/2 | ...... | ...... | 2- 5 | 2- 6 | 2- 6 1/2 | 2- 7 1/2 | 2- 8 | 2- 9 | 2- 9 1/2 | 2-10 | 2-11 |
| *5 3/4 | ...... | ...... | 2- 6 | 2- 7 | 2- 7 1/2 | 2- 8 1/2 | 2- 9 | 2-10 | 2-10 1/2 | 2-11 1/2 | 3- 0 |
| 6 | ...... | ...... | 2- 7 | 2- 8 | 2- 8 1/2 | 2- 9 1/2 | 2-10 | 2-11 | 2-11 1/2 | 3- 0 1/2 | 3- 1 |
| *6 1/4 | ...... | ...... | ...... | 2- 9 | 2- 9 1/2 | 2-10 1/2 | 2-11 | 3- 0 | 3- 0 | 3- 1 1/2 | 3- 2 |
| 6 1/2 | ...... | ...... | ...... | 2-10 | 2-10 1/2 | 2-11 1/2 | 3- 0 | 3- 1 | 3- 1 1/2 | 3- 2 1/2 | 3- 3 |
| *6 3/4 | ...... | ...... | ...... | 2-11 | 3- 0 | 3- 0 1/2 | 3- 1 | 3- 2 | 3- 2 1/2 | 3- 3 1/2 | 3- 4 |
| 7 | ...... | ...... | ...... | 3- 0 | 3- 1 | 3- 1 1/2 | 3-2 1/2 | 3- 3 | 3- 3 1/2 | 3- 4 1/2 | 3- 5 |

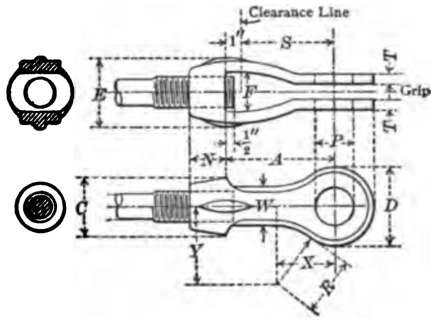
Pins marked * are special. Maximum shipping length of "L" = 35 feet.

## TABLE 22.

### CLEVISES.

**AMERICAN BRIDGE COMPANY STANDARD.**

All dimensions in inches.




Grip = thickness of plate + ¼″.

| Number of Clevis. | Head. Dia. D | Head. Thickness. T | Diameter of Pin, P. Max. | Diameter of Pin, P. Min. | Width. W | Extreme. E | Fork. F | Distance. A | Diameter of Upset. Max. | Diameter of Upset. Min. | Nut. N | Nut. B | Weight, Pounds. | Number of Clevis. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 3 | 3 | ½ | 1½ | 1 | 1½ | 3 1/16 | 1¼ | 5 | 1⅛ | 1 | 1½ | 2¼ | 4 | 3 |
| 4 | 4 | ½ | 2 | 1¼ | 2 | 3⅝ | 1¾ | 6 | 1⅝ | 1⅛ | 1¾ | 2⅞ | 8 | 4 |
| 5 | 5 | ⅝ | 2½ | 1½ | 2½ | 4½ | 2¼ | 7 | 2⅛ | 1½ | 2¼ | 3¾ | 16 | 5 |
| 6 | 6 | ¾ | 3 | 2 | 3 | 5⅜ | 2¾ | 8 | 2½ | 2 | 2½ | 4⅜ | 26 | 6 |
| 7 | 7 | ⅞ | 3½ | 2½ | 3½ | 6 3/16 | 3¼ | 9 | 2⅞ | 2¼ | 3 | 5 | 36 | 7 |

### CLEVIS NUMBERS FOR VARIOUS RODS AND PINS.

| Rods. Round. | Rods. Square. | Rods. Upset. | Pins. 1 | 1¼ | 1½ | 1¾ | 2 | 2¼ | 2½ | 2¾ | 3 | 3¼ | 3½ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| ¾ | ...... | 1 | 3 | 3 | 3 | ...... | ...... | ...... | ...... | ...... | ...... | ...... | ...... |
| ⅞ | ¾ | 1⅛ | 3 | 3 | 3 | 4 | 4 | ...... | ...... | ...... | ...... | ...... | ...... |
| ...... | ⅞ | 1¼ | ...... | 4 | 4 | 4 | 4 | ...... | ...... | ...... | ...... | ...... | ...... |
| 1 | ...... | 1⅜ | ...... | 4 | 4 | 4 | 4 | ...... | ...... | ...... | ...... | ...... | ...... |
| 1⅛ | 1 | 1½ | ...... | 4 | 4 | 4 | 4 | 5 | 5 | ...... | ...... | ...... | ...... |
| 1¼ | 1⅛ | 1⅝ | ...... | 4 | 4 | 4 | 4 | 5 | 5 | ...... | ...... | ...... | ...... |
| 1⅜ | ...... | 1¾ | ...... | ...... | 5 | 5 | 5 | 5 | 5 | ...... | ...... | ...... | ...... |
| 1½ | 1¼ | 1⅞ | ...... | ...... | 5 | 5 | 5 | 5 | 5 | ...... | ...... | ...... | ...... |
| 1⅝ | 1⅜ | 2 | ...... | ...... | 5 | 5 | 5 | 5 | 5 | 6 | 6 | ...... | ...... |
| 1¾ | ...... | 2⅛ | ...... | ...... | 5 | 5 | 5 | 5 | 5 | 6 | 6 | ...... | ...... |
| 1⅞ | 1½ | 2¼ | ...... | ...... | ...... | ...... | 6 | 6 | 6 | 6 | 6 | 7 | 7 |
| 2 | 1⅝ | 2⅜ | ...... | ...... | ...... | ...... | 6 | 6 | 6 | 6 | 6 | 7 | 7 |
| 2⅛ | 1¾ | 2½ | ...... | ...... | ...... | ...... | 6 | 6 | 6 | 6 | 6 | 7 | 7 |
| 2¼ | 1⅞ | 2¾ | ...... | ...... | ...... | ...... | ...... | ...... | 7 | 7 | 7 | 7 | 7 |
| 2⅜ | 2 | 2⅞ | ...... | ...... | ...... | ...... | ...... | ...... | 7 | 7 | 7 | 7 | 7 |

Clevises to be used with the Rods and Pins given above.

Clevises above and to right of zigzag line may be used with forks straight, those below and to left of this line should have forks closed so as not to overstress pin.

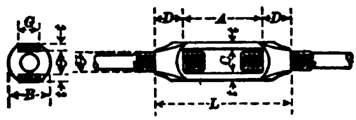
## TABLE 23.

### Turnbuckles and Sleeve Nuts.

AMERICAN BRIDGE COMPANY STANDARD.

All Dimensions in Inches.

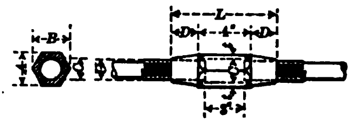
### TURNBUCKLES.



A = 6″; A = 9″9 for turnbuckles marked *.
Pitch and shape of thread, A. B. Co. Standard.

| Diam. of Screw. U | D | L | C | t | G | B | Weight, Pounds. |
|---|---|---|---|---|---|---|---|
| 3/8 | 9/16 | 7 1/8 | 9/16 | 3/16 | 1/2 | 1 1/16 | 1 |
| 7/16 | 21/32 | 7 5/16 | 5/8 | 1/4 | 5/8 | 1 3/8 | 1 |
| 1/2 | 3/4 | 7 1/2 | 5/8 | 1/4 | 5/8 | 1 3/8 | 1 |
| 9/16 | 27/32 | 7 11/16 | 13/16 | 5/16 | 3/4 | 1 9/16 | 1 1/2 |
| 5/8 | 13/16 | 7 7/8 | 13/16 | 5/16 | 3/4 | 1 9/16 | 1 1/2 |
| 3/4 | 1 1/8 | 8 1/4 | 1 1/16 | 11/32 | 7/8 | 2 | 2 |
| 7/8 | 1 5/16 | 8 5/8 | 1 1/4 | 3/8 | 1 | 2 1/4 | 3 |
| 1 | 1 1/2 | 9 | 1 5/16 | 7/16 | 1 1/4 | 2 7/16 | 4 |
| 1 1/8 | 1 11/16 | 9 3/8 | 1 7/16 | 1/2 | 1 1/4 | 2 9/16 | 5 |
| 1 1/4 | 1 7/8 | 9 3/4 | 1 9/16 | 1/2 | 1 1/2 | 2 3/4 | 6 |
| 1 3/8 | 2 1/16 | 10 1/8 | 1 11/16 | 1/2 | 1 5/8 | 3 1/16 | 7 |
| 1 1/2 | 2 1/4 | 10 1/2 | 1 3/4 | 5/8 | 1 3/4 | 3 3/16 | 8 |
| 1 5/8 | 2 7/16 | 10 7/8 | 2 | 5/8 | 1 7/8 | 3 1/2 | 10 |
| 1 3/4 | 2 3/8 | 11 1/4 | 2 1/8 | 5/8 | 2 | 3 3/4 | 11 |
| 1 7/8 | 2 13/16 | 11 5/8 | 2 3/16 | 11/16 | 2 1/8 | 3 7/8 | 12 |
| 2 | 3 | 12 | 2 3/8 | 11/16 | 2 1/4 | 4 1/4 | 14 |
| 2 1/8 | 3 3/16 | 12 3/8 | 2 1/2 | 23/32 | 2 1/2 | 4 1/2 | 17 |
| 2 1/4 | 3 3/8 | 12 3/4 | 2 11/16 | 13/16 | 2 1/2 | 4 3/4 | 20 |
| 2 3/8 | 3 9/16 | 13 1/8 | 2 3/4 | 13/16 | 2 3/4 | 4 7/8 | 22 |
| 2 1/2 | 3 3/4 | 13 1/2 | 3 1/16 | 27/32 | 3 | 5 3/8 | 25 |
| 2 3/4 | 4 1/8 | 14 1/4 | 3 1/4 | 15/16 | 3 1/4 | 5 3/4 | 33 |
| 2 7/8 | 4 5/16 | 14 5/8 | 3 7/16 | 1 1/32 | 3 1/4 | 6 1/16 | 36 |
| 3 | 4 1/2 | 15 | 3 5/8 | 1 1/32 | 3 1/2 | 6 3/8 | 40 |
| 3 1/4 | 4 7/8 | 15 3/4 | 3 7/8 | 1 1/16 | 4 | 6 3/4 | 50 |
| 3 1/2 | 5 1/4 | 16 1/2 | 4 1/4 | 1 7/32 | 4 | 7 1/4 | 65 |
| 3 3/4 | 5 5/8 | 17 1/4 | 4 7/16 | 1 5/16 | 5 | 8 1/4 | 95 |
| 4 | 6 | 18 | 4 5/8 | 1 7/16 | 5 | 8 3/4 | 108 |
| *4 1/4 | 6 1/4 | 21 1/2 | 4 5/8 | 1 5/8 | 5 5/32 | 9 1/4 | 140 |
| *4 1/2 | 6 3/4 | 22 1/2 | 5 1/2 | 1 3/4 | 6 1/2 | 10 3/4 | 195 |
| *4 3/4 | 7 1/4 | 23 1/2 | 5 5/8 | 2 | 6 1/2 | 11 1/4 | 205 |
| *5 | 7 1/2 | 24 | 6 | 2 1/4 | 6 1/2 | 11 7/8 | 250 |

### SLEEVE NUTS.



Pitch and shape of thread, A. B. Co. Standard.

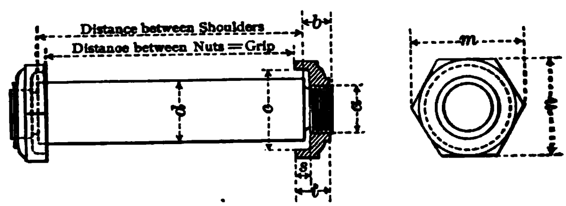
| Diam. of Screw. U | D | L | A | B | C | t | Weight, Pounds. |
|---|---|---|---|---|---|---|---|
| 7/8 | 1 1/2 | 7 | 1 5/8 | 1 7/8 | 1 1/8 | 1/4 | 3 |
| 1 | 1 1/2 | 7 | 1 5/8 | 1 7/8 | 1 1/8 | 1/4 | 3 |
| 1 1/8 | 1 3/4 | 7 1/2 | 2 | 2 5/16 | 1 3/8 | 5/16 | 4 |
| 1 1/4 | 1 3/4 | 7 1/2 | 2 | 2 5/16 | 1 3/8 | 5/16 | 4 |
| 1 3/8 | 2 | 8 | 2 3/8 | 2 3/4 | 1 5/8 | 3/8 | 5 |
| 1 1/2 | 2 | 8 | 2 3/8 | 2 3/4 | 1 5/8 | 3/8 | 6 |
| 1 5/8 | 2 1/4 | 8 1/2 | 2 3/4 | 3 5/16 | 1 7/8 | 7/16 | 8 |
| 1 3/4 | 2 1/4 | 8 1/2 | 2 3/4 | 3 5/16 | 1 7/8 | 7/16 | 9 |
| 1 7/8 | 2 1/2 | 9 | 3 1/8 | 3 5/8 | 2 1/8 | 1/2 | 10 |
| 2 | 2 1/2 | 9 | 3 1/8 | 3 3/8 | 2 1/8 | 1/2 | 11 |
| 2 1/8 | 2 3/4 | 9 1/2 | 3 1/2 | 4 1/16 | 2 3/8 | 9/16 | 14 |
| 2 1/4 | 2 3/4 | 9 1/2 | 3 1/2 | 4 1/16 | 2 3/8 | 9/16 | 15 |
| 2 3/8 | 3 | 10 | 3 7/8 | 4 1/2 | 2 5/8 | 5/8 | 18 |
| 2 1/2 | 3 | 10 | 3 7/8 | 4 1/2 | 2 5/8 | 5/8 | 19 |
| 2 3/4 | 3 1/4 | 10 1/2 | 4 1/4 | 4 15/16 | 2 7/8 | 11/16 | 23 |
| 2 7/8 | 3 1/2 | 11 | 4 5/8 | 5 3/8 | 3 1/8 | 3/4 | 27 |
| 3 | 3 1/2 | 11 | 4 5/8 | 5 3/8 | 3 1/8 | 3/4 | 28 |
| 3 1/4 | 3 3/4 | 11 1/2 | 5 | 5 13/16 | 3 3/8 | 13/16 | 35 |
| 3 1/2 | 4 | 12 | 5 3/8 | 6 1/4 | 3 5/8 | 7/8 | 40 |
| 3 3/4 | 4 1/4 | 12 1/2 | 5 3/4 | 6 11/16 | 3 7/8 | 15/16 | 47 |
| 4 | 4 1/2 | 13 | 6 1/8 | 7 1/16 | 4 1/8 | 1 | 55 |
| 4 1/4 | 4 3/4 | 13 1/2 | 6 1/2 | 7 1/2 | 4 3/8 | 1 1/16 | 65 |
| 4 1/2 | 5 | 14 | 6 7/8 | 7 15/16 | 4 3/4 | 1 1/16 | 75 |

## TABLE 24.

### Bridge Pins and Nuts.

AMERICAN BRIDGE COMPANY STANDARD.

All Dimensions in Inches.




To obtain grip, add 1/16″ for each bar. Nuts threaded 6 threads per inch.
To obtain distance between shoulders, add amount given in table to grip.

| Diameter of Pin, d. | Pin. Thread. a | Pin. Thread. b | Pin. Add to Grip. | Nut. Thickness. t | Nut. Diameter. n | Nut. Diameter. m | Nut. Diameter. c | Nut. Depth s. | Nut. Diameter Rough Hole. | Weight, Pounds. | Pattern No. |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 2, 2¼ | 1½ | 1 | ¼ | ⅞ | 2 15/16 | 3⅜ | 2⅝ | ¼ | 1 5/16 | 1.1 | PN 21 |
| 2½, 2¾ | 2 | 1⅛ | ¼ | 1 | 3 9/16 | 4⅛ | 3⅛ | ¼ | 1 13/16 | 1.7 | PN 22 |
| 3, *3¼, 3½ | 2½ | 1¼ | ¼ | 1⅛ | 4 5/16 | 5 | 3⅞ | ⅜ | 2 5/16 | 2.5 | PN 23 |
| *3¾, 4 | 3 | 1⅜ | ½ | 1¼ | 4⅞ | 5⅝ | 4⅜ | ⅜ | 2 13/16 | 3.7 | PN 24 |
| *4¼, 4½, *4¾ | 3½ | 1½ | ½ | 1⅜ | 5¾ | 6⅝ | 5¼ | ½ | 3 5/16 | 4.6 | PN 25 |
| 5, *5¼ | 4 | 1⅝ | ½ | 1½ | 6¼ | 7 3/16 | 5¾ | ½ | 3 13/16 | 6.2 | PN 26 |
| 5½, *5¾, 6 | 4½ | 1¾ | ½ | 1⅝ | 7 | 8⅛ | 6½ | ⅝ | 4 5/16 | 7.8 | PN 27 |
| *6¼, *6½ | 5 | 1⅞ | ¾ | 1¾ | 7⅝ | 8⅞ | 7 | ⅝ | 4 13/16 | 9.9 | PN 28 |
| *6¾, 7 | 5½ | 2 | ¾ | 1⅞ | 8⅛ | 9⅜ | 7½ | ¾ | 5 5/16 | 11.8 | PN 29 |
| *7¼, *7½ | 5½ | 2 | ¾ | 1⅞ | 8⅝ | 10 | 8 | ¾ | 5 5/16 | 14.3 | PN 30 |
| *7¾, 8, *8¼ | 6 | 2¼ | ¾ | 2⅛ | 9⅝ | 10⅞ | 8¾ | ¾ | 5 13/16 | 18.6 | PN 31 |
| *8½, 9 | 6 | 2¼ | ¾ | 2⅛ | 10¼ | 11⅞ | 9⅝ | ¾ | 5 13/16 | 23.8 | PN 32 |
| *9½, 10 | 6 | 2⅜ | ¾ | 2¼ | 11¼ | 13 | 10⅝ | ¾ | 5 13/16 | 31.1 | PN 33 |

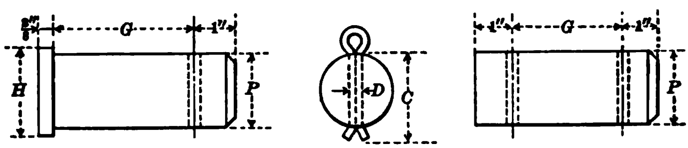
Pins marked * are special.

## TABLE 25.

### Cotter Pins.

AMERICAN BRIDGE COMPANY STANDARD.

All Dimensions in Inches.




| Horizontal or Vertical Pin Finished. | | | | | Horizontal Pin Rough or Finished. | | | |
|---|---|---|---|---|---|---|---|---|
| Pin. | Head. | G | Cotter. | | Pin. | G | Cotter. | |
| P | H | | C | D | P | | C | D |
| $1\frac{1}{4}$ | $1\frac{1}{2}$ | Net Grip + $\frac{1}{2}''$ | 2 | $\frac{1}{4}$ | $1\frac{1}{4}$ | Net Grip + $\frac{3}{4}''$ | 2 | $\frac{1}{4}$ |
| $1\frac{1}{2}$ | $1\frac{3}{4}$ | | $2\frac{1}{2}$ | $\frac{1}{4}$ | $1\frac{1}{2}$ | | $2\frac{1}{2}$ | $\frac{1}{4}$ |
| $1\frac{3}{4}$ | 2 | | $2\frac{3}{4}$ | $\frac{1}{4}$ | $1\frac{3}{4}$ | | $2\frac{3}{4}$ | $\frac{1}{4}$ |
| 2 | $2\frac{3}{8}$ | | 3 | $\frac{3}{8}$ | 2 | | 3 | $\frac{3}{8}$ |
| $2\frac{1}{4}$ | $2\frac{5}{8}$ | | $3\frac{1}{4}$ | $\frac{3}{8}$ | $2\frac{1}{4}$ | | $3\frac{1}{4}$ | $\frac{3}{8}$ |
| $2\frac{1}{2}$ | $2\frac{7}{8}$ | | $3\frac{3}{4}$ | $\frac{3}{8}$ | $2\frac{1}{2}$ | | $3\frac{3}{4}$ | $\frac{3}{8}$ |
| $2\frac{3}{4}$ | $3\frac{1}{8}$ | | 4 | $\frac{3}{8}$ | $2\frac{3}{4}$ | | 4 | $\frac{3}{8}$ |
| 3 | $3\frac{3}{8}$ | | 5 | $\frac{1}{2}$ | 3 | | 5 | $\frac{1}{2}$ |
| $3\frac{1}{4}$ | $3\frac{3}{4}$ | | 5 | $\frac{1}{2}$ | $3\frac{1}{4}$ | | 5 | $\frac{1}{2}$ |
| $3\frac{1}{2}$ | 4 | | 6 | $\frac{1}{2}$ | $3\frac{1}{2}$ | | 6 | $\frac{1}{2}$ |
| $3\frac{3}{4}$ | $4\frac{1}{4}$ | | 6 | $\frac{1}{2}$ | $3\frac{3}{4}$ | | 6 | $\frac{1}{2}$ |

## TABLE 26

### Bearing Values of Pins.

| Pin. | | Bearing Value of Plate 1″ Thick for Unit Stress per Square Inch of | | | | | Diam. of Pin in In. |
|---|---|---|---|---|---|---|---|
| Diam. in In. | Area. | 12 000 | 15 000 | 20 000 | 22 000 | 24 000 | |
| 1 | .785 | 12 000 | 15 000 | 20 000 | 22 000 | 24 000 | 1 |
| 1¼ | 1.227 | 15 000 | 18 800 | 25 000 | 27 500 | 30 000 | 1¼ |
| 1½ | 1.767 | 18 000 | 22 500 | 30 000 | 33 000 | 36 000 | 1½ |
| 1¾ | 2.405 | 21 000 | 26 300 | 35 000 | 38 500 | 42 000 | 1¾ |
| 2 | 3.142 | 24 000 | 30 000 | 40 000 | 44 000 | 48 000 | 2 |
| 2¼ | 3.976 | 27 000 | 33 800 | 45 000 | 49 500 | 54 000 | 2¼ |
| 2½ | 4.909 | 30 000 | 37 500 | 50 000 | 55 000 | 60 000 | 2½ |
| 2¾ | 5.940 | 33 000 | 41 300 | 55 000 | 60 500 | 66 000 | 2¾ |
| 3 | 7.069 | 36 000 | 45 000 | 60 000 | 66 000 | 72 000 | 3 |
| 3¼ | 8.296 | 39 000 | 48 800 | 65 000 | 71 500 | 78 000 | 3¼ |
| 3½ | 9.621 | 42 000 | 52 500 | 70 000 | 77 000 | 84 000 | 3½ |
| 3¾ | 11.045 | 45 000 | 56 300 | 75 000 | 82 500 | 90 000 | 3¾ |
| 4 | 12.566 | 48 000 | 60 000 | 80 000 | 88 000 | 96 000 | 4 |
| 4¼ | 14.186 | 51 000 | 63 800 | 85 000 | 93 500 | 102 000 | 4¼ |
| 4½ | 15.904 | 54 000 | 67 500 | 90 000 | 99 000 | 108 000 | 4½ |
| 4¾ | 17.721 | 57 000 | 71 300 | 95 000 | 104 500 | 114 000 | 4¾ |
| 5 | 19.635 | 60 000 | 75 000 | 100 000 | 110 000 | 120 000 | 5 |
| 5¼ | 21.648 | 63 000 | 78 800 | 105 000 | 115 500 | 126 000 | 5¼ |
| 5½ | 23.758 | 66 000 | 82 500 | 110 000 | 121 000 | 132 000 | 5½ |
| 5¾ | 25.967 | 69 000 | 86 300 | 115 000 | 126 500 | 138 000 | 5¾ |
| 6 | 28.274 | 72 000 | 90 000 | 120 000 | 132 000 | 144 000 | 6 |
| 6¼ | 30.680 | 75 000 | 93 800 | 125 000 | 137 500 | 150 000 | 6¼ |
| 6½ | 33.183 | 78 000 | 97 500 | 130 000 | 143 000 | 156 000 | 6½ |
| 6¾ | 35.785 | 81 000 | 101 300 | 135 000 | 148 500 | 162 000 | 6¾ |
| 7 | 38.485 | 84 000 | 105 000 | 140 000 | 154 000 | 168 000 | 7 |
| 7¼ | 41.282 | 87 000 | 108 800 | 145 000 | 159 500 | 174 000 | 7¼ |
| 7½ | 44.179 | 90 000 | 112 500 | 150 000 | 165 000 | 180 000 | 7½ |
| 7¾ | 47.173 | 93 000 | 116 300 | 155 000 | 170 500 | 186 000 | 7¾ |
| 8 | 50.265 | 96 000 | 120 000 | 160 000 | 176 000 | 192 000 | 8 |
| 8¼ | 53.456 | 99 000 | 123 800 | 165 000 | 181 500 | 198 000 | 8¼ |
| 8½ | 56.745 | 102 000 | 127 500 | 170 000 | 187 000 | 204 000 | 8½ |
| 8¾ | 60.132 | 105 000 | 131 300 | 175 000 | 192 500 | 210 000 | 8¾ |
| 9 | 63.617 | 108 000 | 135 000 | 180 000 | 198 000 | 216 000 | 9 |
| 9¼ | 67.201 | 111 000 | 138 800 | 185 000 | 203 500 | 222 000 | 9¼ |
| 9½ | 70.882 | 114 000 | 142 500 | 190 000 | 209 000 | 228 000 | 9½ |
| 9¾ | 74.662 | 117 000 | 146 300 | 195 000 | 214 500 | 234 000 | 9¾ |
| 10 | 78.540 | 120 000 | 150 000 | 200 000 | 220 000 | 240 000 | 10 |
| 10¼ | 82.516 | 123 000 | 153 800 | 205 000 | 225 500 | 246 000 | 10¼ |
| 10½ | 86.590 | 126 000 | 157 500 | 210 000 | 231 000 | 252 000 | 10½ |
| 10¾ | 90.763 | 129 000 | 161 300 | 215 000 | 236 500 | 258 000 | 10¾ |
| 11 | 95.033 | 132 000 | 165 000 | 220 000 | 242 000 | 264 000 | 11 |
| 11¼ | 99.402 | 135 000 | 168 800 | 225 000 | 247 500 | 270 000 | 11¼ |
| 11½ | 103.869 | 138 000 | 172 500 | 230 000 | 253 000 | 276 000 | 11½ |
| 11¾ | 108.434 | 141 000 | 176 300 | 235 000 | 258 500 | 282 000 | 11¾ |
| 12 | 113.097 | 144 000 | 180 000 | 240 000 | 264 000 | 288 000 | 12 |

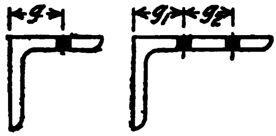
## TABLE 27.

### Bending Moments on Pins.

| Pin. | | Max. Moments in Inch-Pounds for Fiber Stress per Square Inch of | | | | | | | Diam. of Pin in In. |
|---|---|---|---|---|---|---|---|---|---|
| Diam. in In. | Area. | 15 000 | 18 000 | 20 000 | 22 000 | 22 500 | 24 000 | 25 000 | |
| 1 | .785 | 1 470 | 1 770 | 1 960 | 2 160 | 2 210 | 2 360 | 2 450 | 1 |
| 1¼ | 1.227 | 2 880 | 3 450 | 3 830 | 4 220 | 4 310 | 4 600 | 4 790 | 1¼ |
| 1½ | 1.767 | 4 970 | 5 960 | 6 630 | 7 290 | 7 460 | 7 950 | 8 280 | 1½ |
| 1¾ | 2.405 | 7 890 | 9 470 | 10 500 | 11 580 | 11 800 | 12 630 | 13 200 | 1¾ |
| 2 | 3.142 | 11 800 | 14 100 | 15 700 | 17 280 | 17 700 | 18 800 | 19 600 | 2 |
| 2¼ | 3.976 | 16 800 | 20 100 | 22 400 | 24 600 | 25 200 | 26 800 | 28 000 | 2¼ |
| 2½ | 4.909 | 23 000 | 27 600 | 30 700 | 33 700 | 34 500 | 36 800 | 38 300 | 2½ |
| 2¾ | 5.940 | 30 600 | 36 800 | 40 800 | 44 900 | 45 900 | 49 000 | 51 000 | 2¾ |
| 3 | 7.069 | 39 800 | 47 700 | 53 000 | 58 300 | 59 600 | 63 600 | 66 300 | 3 |
| 3¼ | 8.296 | 50 600 | 60 700 | 67 400 | 74 100 | 75 800 | 80 900 | 84 300 | 3¼ |
| 3½ | 9.621 | 63 100 | 75 800 | 84 200 | 92 600 | 94 700 | 101 000 | 105 200 | 3½ |
| 3¾ | 11.045 | 77 700 | 93 200 | 103 500 | 113 900 | 116 500 | 124 300 | 129 400 | 3¾ |
| 4 | 12.566 | 94 200 | 113 100 | 125 700 | 138 200 | 141 400 | 150 800 | 157 100 | 4 |
| 4¼ | 14.186 | 113 000 | 135 700 | 150 700 | 165 800 | 169 600 | 180 900 | 188 400 | 4¼ |
| 4½ | 15.904 | 134 200 | 161 000 | 178 900 | 196 800 | 201 300 | 214 700 | 223 700 | 4½ |
| 4¾ | 17.721 | 157 800 | 189 400 | 210 400 | 231 500 | 236 700 | 252 500 | 263 000 | 4¾ |
| 5 | 19.635 | 184 100 | 220 900 | 245 400 | 270 000 | 276 100 | 294 500 | 306 800 | 5 |
| 5¼ | 21.648 | 213 100 | 255 700 | 284 100 | 312 500 | 319 600 | 340 900 | 355 200 | 5¼ |
| 5½ | 23.758 | 245 000 | 294 000 | 326 700 | 359 300 | 367 500 | 392 000 | 408 300 | 5½ |
| 5¾ | 25.967 | 280 000 | 336 000 | 373 300 | 410 600 | 419 900 | 447 900 | 466 600 | 5¾ |
| 6 | 28.274 | 318 100 | 381 700 | 424 100 | 466 500 | 477 100 | 508 900 | 530 100 | 6 |
| 6¼ | 30.680 | 359 500 | 431 400 | 479 400 | 527 300 | 539 300 | 575 200 | 599 200 | 6¼ |
| 6½ | 33.183 | 404 400 | 485 300 | 539 200 | 593 100 | 606 600 | 647 100 | 674 000 | 6½ |
| 6¾ | 35.785 | 452 900 | 543 500 | 603 900 | 664 300 | 679 400 | 724 600 | 754 800 | 6¾ |
| 7 | 38.485 | 505 100 | 606 100 | 673 500 | 740 800 | 757 700 | 808 200 | 841 800 | 7 |
| 7¼ | 41.282 | 561 200 | 673 400 | 748 200 | 823 100 | 841 800 | 897 900 | 935 300 | 7¼ |
| 7½ | 44.179 | 621 300 | 745 500 | 828 400 | 911 200 | 931 900 | 994 000 | 1 035 400 | 7½ |
| 7¾ | 47.173 | 685 500 | 822 600 | 914 000 | 1 005 400 | 1 028 200 | 1 096 800 | 1 142 500 | 7¾ |
| 8 | 50.265 | 754 000 | 904 800 | 1 005 300 | 1 105 800 | 1 131 000 | 1 206 400 | 1 256 600 | 8 |
| 8¼ | 53.456 | 826 900 | 992 300 | 1 102 500 | 1 212 800 | 1 240 400 | 1 323 000 | 1 378 200 | 8¼ |
| 8½ | 56.745 | 904 400 | 1 085 300 | 1 205 800 | 1 326 400 | 1 356 600 | 1 447 000 | 1 507 300 | 8½ |
| 8¾ | 60.132 | 986 500 | 1 183 900 | 1 315 400 | 1 446 900 | 1 479 800 | 1 578 500 | 1 644 200 | 8¾ |
| 9 | 63.617 | 1 073 500 | 1 288 200 | 1 431 400 | 1 574 500 | 1 610 300 | 1 717 700 | 1 789 200 | 9 |
| 9¼ | 67.201 | 1 165 500 | 1 398 600 | 1 554 000 | 1 709 400 | 1 748 300 | 1 864 800 | 1 942 500 | 9¼ |
| 9½ | 70.882 | 1 262 600 | 1 515 100 | 1 683 500 | 1 851 800 | 1 893 900 | 2 020 100 | 2 104 300 | 9½ |
| 9¾ | 74.662 | 1 364 900 | 1 637 900 | 1 819 900 | 2 001 900 | 2 047 400 | 2 183 900 | 2 274 900 | 9¾ |
| 10 | 78.540 | 1 472 600 | 1 767 100 | 1 963 500 | 2 159 800 | 2 208 900 | 2 356 200 | 2 454 400 | 10 |
| 10¼ | 82.516 | 1 585 900 | 1 903 000 | 2 114 500 | 2 325 900 | 2 378 800 | 2 537 400 | 2 643 100 | 10¼ |
| 10½ | 86.590 | 1 704 700 | 2 045 700 | 2 273 000 | 2 500 300 | 2 557 100 | 2 727 600 | 2 841 200 | 10½ |
| 10¾ | 90.763 | 1 829 400 | 2 195 300 | 2 439 200 | 2 683 200 | 2 744 100 | 2 927 100 | 3 049 100 | 10¾ |
| 11 | 95.033 | 1 960 100 | 2 352 100 | 2 613 400 | 2 874 800 | 2 940 100 | 3 136 100 | 3 266 800 | 11 |
| 11¼ | 99.402 | 2 096 800 | 2 516 100 | 2 795 700 | 3 075 200 | 3 145 100 | 3 354 800 | 3 494 600 | 11¼ |
| 11½ | 103.869 | 2 239 700 | 2 687 600 | 2 986 200 | 3 284 900 | 3 359 500 | 3 583 500 | 3 732 800 | 11½ |
| 11¾ | 108.434 | 2 388 900 | 2 866 700 | 3 185 200 | 3 503 800 | 3 583 400 | 3 822 300 | 3 981 600 | 11¾ |
| 12 | 113.097 | 2 544 700 | 3 053 600 | 3 392 900 | 3 732 200 | 3 817 000 | 4 071 500 | 4 241 200 | 12 |

## TABLE 28.

### Standards for Rivets and Riveting.



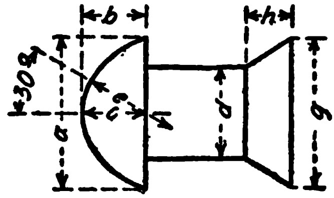

**GAGES in inches**

| Leg | Gage $g$ | Max. Rivet |
|---|---|---|
| 8 | $4\frac{1}{2}$ | $\frac{7}{8}$ |
| 7 | 4 | $\frac{7}{8}$ |
| 6 | $3\frac{1}{2}$ | $\frac{7}{8}$ |
| 5 | 3 | $\frac{7}{8}$ |
| 4 | $2\frac{1}{2}$ | $\frac{7}{8}$ |
| $3\frac{1}{2}$ | 2 | $\frac{7}{8}$ |
| 3 | $1\frac{3}{4}$ | $\frac{7}{8}$ |
| $2\frac{3}{4}$ | $1\frac{5}{8}$ | $\frac{3}{4}$ |
| $2\frac{1}{2}$ | $1\frac{3}{8}$ | $\frac{3}{4}$ |
| $2\frac{1}{4}$ | $1\frac{1}{4}$ | $\frac{5}{8}$ |
| 2 | $1\frac{1}{8}$ | $\frac{5}{8}$ |
| $1\frac{3}{4}$ | 1 | $\frac{1}{2}$ |
| $1\frac{1}{2}$ | $\frac{7}{8}$ | $\frac{3}{8}$ |
| $1\frac{3}{8}$ | $\frac{7}{8}$ | $\frac{3}{8}$ |
| $1\frac{1}{4}$ | $\frac{3}{4}$ | $\frac{3}{8}$ |
| 1 | $\frac{5}{8}$ | $\frac{1}{4}$ |

| Leg | Gage $g_1$ | Gage $g_2$ | Max. Rivet |
|---|---|---|---|
| 8 | 3 | 3 | $\frac{7}{8}$ |
| 7 | $2\frac{1}{2}$ | 3 | $\frac{7}{8}$ |
| 6 | $2\frac{1}{2}$ | $2\frac{1}{4}$ | $\frac{7}{8}$ |
| 5 | 2 | $1\frac{3}{4}$ | $\frac{7}{8}$ |
| When 6" L exceeds $\frac{3}{4}$" | | | |
| 6 | $2\frac{1}{2}$ | 2 | $\frac{7}{8}$ |

**MINIMUM RIVET SPACING**

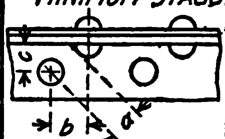
| Size of Rivet inches | Min. Distance inches |
|---|---|
| 1 | 3 |
| $\frac{7}{8}$ | $2\frac{5}{8}$ |
| $\frac{3}{4}$ | $2\frac{1}{4}$ |
| $\frac{5}{8}$ | 2 |
| $\frac{1}{2}$ | $1\frac{3}{4}$ |
| $\frac{3}{8}$ | $1\frac{1}{4}$ |
| $\frac{1}{4}$ | 1 |




**PROPORTIONS OF RIVETS in inches**

| Diameter of Shank $d$ | Full Head: Diameter $a$ | Full Head: Height $b$ | Full Head: Radii $c$ | Full Head: Radii $e$ | Countersunk: Diameter $g$ | Countersunk: Depth $h$ |
|---|---|---|---|---|---|---|
| 1 | $1\frac{5}{8}$ | $\frac{11}{16}$ | $\frac{11}{16}$ | $1\frac{1}{32}$ | $1\frac{9}{16}$ | $\frac{1}{2}$ |
| $\frac{7}{8}$ | $1\frac{7}{16}$ | $\frac{39}{64}$ | $\frac{39}{64}$ | $\frac{59}{64}$ | $1\frac{3}{8}$ | $\frac{7}{16}$ |
| $\frac{3}{4}$ | $1\frac{1}{4}$ | $\frac{17}{32}$ | $\frac{17}{32}$ | $\frac{51}{64}$ | $1\frac{3}{16}$ | $\frac{3}{8}$ |
| $\frac{5}{8}$ | $1\frac{1}{16}$ | $\frac{29}{64}$ | $\frac{29}{64}$ | $\frac{43}{64}$ | 1 | $\frac{5}{16}$ |
| $\frac{1}{2}$ | $\frac{7}{8}$ | $\frac{3}{8}$ | $\frac{3}{8}$ | $\frac{9}{16}$ | $\frac{25}{32}$ | $\frac{1}{4}$ |
| $\frac{3}{8}$ | $\frac{11}{16}$ | $\frac{19}{64}$ | $\frac{19}{64}$ | $\frac{7}{16}$ | $\frac{19}{32}$ | $\frac{3}{16}$ |

**MINIMUM STAGGER FOR RIVETS**

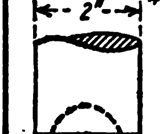



| $c$ inches | $b$ in inches, For $\frac{3}{4}$" Rivet $a = 1\frac{1}{8}$" | $b$ in inches, For $\frac{7}{8}$" Rivet $a = 1\frac{1}{4}$" |
|---|---|---|
| $1\frac{1}{8}$ | $1\frac{1}{4}$ | $1\frac{1}{2}$ |
| $1\frac{3}{16}$ | $1\frac{3}{16}$ | $1\frac{7}{16}$ |
| $1\frac{1}{4}$ | $1\frac{1}{8}$ | $1\frac{3}{8}$ |
| $1\frac{5}{16}$ | $1\frac{1}{16}$ | $1\frac{5}{16}$ |
| $1\frac{3}{8}$ | $\frac{15}{16}$ | $1\frac{1}{4}$ |
| $1\frac{7}{16}$ | $\frac{7}{8}$ | $1\frac{3}{16}$ |
| $1\frac{1}{2}$ | $\frac{3}{4}$ | $1\frac{1}{8}$ |
| $1\frac{9}{16}$ | $\frac{9}{16}$ | 1 |
| $1\frac{5}{8}$ | $\frac{3}{8}$ | $\frac{15}{16}$ |
| $1\frac{11}{16}$ | 0 | $\frac{13}{16}$ |
| $1\frac{3}{4}$ | | $\frac{5}{8}$ |
| $1\frac{13}{16}$ | | $\frac{7}{16}$ |
| $1\frac{7}{8}$ | | 0 |

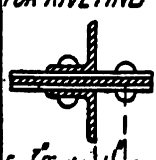
**MINIMUM BUTTON SETS**

For $\frac{3}{4}$" and $\frac{7}{8}$" Rivets $2\frac{1}{4}$"

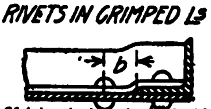
For Rivets less than $\frac{3}{4}$" 2"



**STANDARD CLEARANCE FOR RIVETING**




**RIVETS IN CRIMPED Ls**

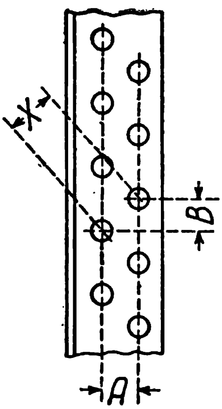



Distance b should $= 1\frac{1}{2}$" + 2 × thickness of crimped Ls, but never less than 2".

## TABLE 29.

### Standards for Riveting.

## Distance ¢ to ¢ of Staggered Rivets.




**Values of X for varying values of A and B.**

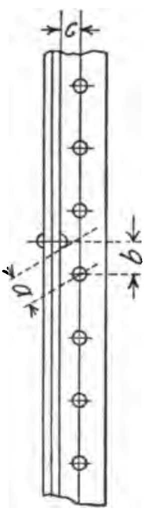
| Values of B | Values of A: 7/8 | 1 | 1 1/8 | 1 1/4 | 1 3/8 | 1 1/2 | 1 5/8 | 1 3/4 | 1 7/8 | 2 | 2 1/8 | 2 1/4 | 2 3/8 | 2 1/2 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 1/8 | 1 7/16 | 1 1/2 | 1 9/16 | 1 11/16 | 1 3/4 | 1 7/8 | 2 | 2 1/16 | 2 3/16 | 2 5/16 | 2 3/8 | 2 1/2 | 2 5/8 | 2 3/4 |
| 1 1/4 | 1 9/16 | 1 5/8 | 1 11/16 | 1 3/4 | 1 7/8 | 1 15/16 | 2 1/16 | 2 1/8 | 2 1/4 | 2 3/8 | 2 7/16 | 2 9/16 | 2 11/16 | 2 13/16 |
| 1 3/8 | 1 5/8 | 1 11/16 | 1 3/4 | 1 7/8 | 1 15/16 | 2 | 2 1/8 | 2 3/16 | 2 5/16 | 2 7/16 | 2 1/2 | 2 5/8 | 2 3/4 | 2 7/8 |
| 1 1/2 | 1 3/4 | 1 13/16 | 1 7/8 | 1 15/16 | 2 | 2 1/8 | 2 3/16 | 2 5/16 | 2 3/8 | 2 1/2 | 2 5/8 | 2 11/16 | 2 13/16 | 2 15/16 |
| 1 5/8 | 1 7/8 | 1 7/8 | 2 | 2 1/16 | 2 1/8 | 2 3/16 | 2 5/16 | 2 3/8 | 2 1/2 | 2 9/16 | 2 11/16 | 2 3/4 | 2 7/8 | 3 |
| 1 3/4 | 1 15/16 | 2 | 2 1/16 | 2 1/8 | 2 3/16 | 2 5/16 | 2 3/8 | 2 7/16 | 2 9/16 | 2 5/8 | 2 3/4 | 2 7/8 | 2 15/16 | 3 1/16 |
| 1 7/8 | 2 1/16 | 2 1/8 | 2 3/16 | 2 1/4 | 2 5/16 | 2 3/8 | 2 1/2 | 2 9/16 | 2 5/8 | 2 3/4 | 2 13/16 | 2 15/16 | 3 | 3 1/8 |
| 2 | 2 3/16 | 2 1/4 | 2 5/16 | 2 3/8 | 2 7/16 | 2 1/2 | 2 9/16 | 2 5/8 | 2 3/4 | 2 13/16 | 2 15/16 | 3 | 3 1/8 | 3 3/16 |
| 2 1/8 | 2 5/16 | 2 5/16 | 2 3/8 | 2 7/16 | 2 1/2 | 2 5/8 | 2 11/16 | 2 3/4 | 2 13/16 | 2 15/16 | 3 | 3 1/16 | 3 3/16 | 3 1/4 |
| 2 1/4 | 2 7/16 | 2 7/16 | 2 1/2 | 2 9/16 | 2 5/8 | 2 11/16 | 2 3/4 | 2 7/8 | 2 15/16 | 3 | 3 1/16 | 3 3/16 | 3 1/4 | 3 3/8 |
| 2 3/8 | 2 1/2 | 2 9/16 | 2 5/8 | 2 11/16 | 2 3/4 | 2 13/16 | 2 7/8 | 2 15/16 | 3 | 3 1/8 | 3 3/16 | 3 1/4 | 3 3/8 | 3 7/16 |
| 2 1/2 | 2 5/8 | 2 11/16 | 2 3/4 | 2 13/16 | 2 7/8 | 2 15/16 | 3 | 3 1/16 | 3 1/8 | 3 3/16 | 3 1/4 | 3 3/8 | 3 7/16 | 3 9/16 |

NOTE:—Values below or to the right of upper zigzag line are large enough for 5/8" Riv.
" " " " " " " second " " " " " " 3/4" "
" " " " " " " lower " " " " " " 7/8" "

## TABLE 30.

### Standards for Riveting.

### Spacing of Staggered Rivets in Angles

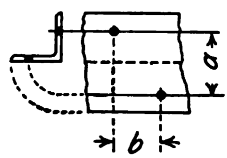



| c in inches | b in inches 5/8" riv. | 3/4" riv. | 7/8" riv. |
|---|---|---|---|
| $1\frac{1}{8}$ | $\frac{15}{16}$ | $1\frac{1}{4}$ | $1\frac{1}{2}$ |
| $1\frac{3}{16}$ | $\frac{7}{8}$ | $1\frac{3}{16}$ | $1\frac{7}{16}$ |
| $1\frac{1}{4}$ | $\frac{13}{16}$ | $1\frac{1}{8}$ | $1\frac{3}{8}$ |
| $1\frac{5}{16}$ | $\frac{11}{16}$ | $1\frac{1}{16}$ | $1\frac{5}{16}$ |
| $1\frac{3}{8}$ | $\frac{1}{2}$ | $\frac{15}{16}$ | $1\frac{1}{4}$ |
| $1\frac{7}{16}$ | $\frac{5}{16}$ | $\frac{7}{8}$ | $1\frac{3}{16}$ |
| $1\frac{1}{2}$ | 0 | $\frac{3}{4}$ | $1\frac{1}{8}$ |
| $1\frac{9}{16}$ | | $\frac{9}{16}$ | 1 |
| $1\frac{5}{8}$ | | $\frac{3}{8}$ | $\frac{15}{16}$ |
| $1\frac{11}{16}$ | | 0 | $\frac{13}{16}$ |
| $1\frac{3}{4}$ | | | $\frac{5}{8}$ |
| $1\frac{13}{16}$ | | | $\frac{7}{16}$ |
| $1\frac{7}{8}$ | | | 0 |

a = 1" for $\frac{5}{8}$" rivets; $1\frac{1}{8}$" for $\frac{3}{4}$" rivets; $1\frac{1}{4}$" for $\frac{7}{8}$" rivets

### Stagger of Rivets Required to Maintain Net Section

ONE HOLE OUT

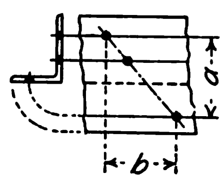



$y$ = diam. of riv. + $\frac{1}{8}$"

$a - y = \sqrt{a^2 + b^2} - 2y$

$b = \sqrt{2ay + y^2}$

TWO HOLES OUT




$y$ = diam. of riv. + $\frac{1}{8}$"

$a - 2y = \sqrt{a^2 + b^2} - 3y$

$b = \sqrt{2ay + y^2}$

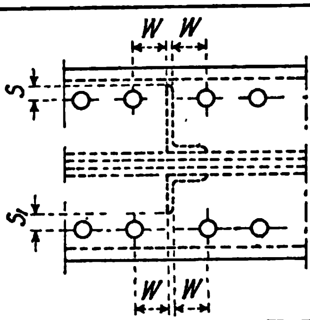
| Sum of Gages | Size of Rivet 3/4" | 7/8" |
|---|---|---|
| a | b | c |
| 1 | $1\frac{5}{8}$ | $1\frac{3}{4}$ |
| $1\frac{1}{2}$ | $1\frac{7}{8}$ | 2 |
| 2 | $2\frac{1}{16}$ | $2\frac{1}{4}$ |
| $2\frac{1}{2}$ | $2\frac{1}{4}$ | $2\frac{7}{16}$ |
| 3 | $2\frac{7}{16}$ | $2\frac{5}{8}$ |
| $3\frac{1}{2}$ | $2\frac{9}{16}$ | $2\frac{13}{16}$ |
| 4 | $2\frac{13}{16}$ | 3 |
| $4\frac{1}{2}$ | $2\frac{15}{16}$ | $3\frac{3}{16}$ |
| 5 | $3\frac{1}{16}$ | $3\frac{5}{16}$ |
| $5\frac{1}{2}$ | $3\frac{1}{4}$ | $3\frac{1}{2}$ |
| 6 | $3\frac{3}{8}$ | $3\frac{5}{8}$ |
| $6\frac{1}{2}$ | $3\frac{1}{2}$ | $3\frac{3}{4}$ |
| 7 | $3\frac{5}{8}$ | $3\frac{7}{8}$ |
| $7\frac{1}{2}$ | $3\frac{3}{4}$ | 4 |
| 8 | $3\frac{7}{8}$ | $4\frac{1}{8}$ |
| $8\frac{1}{2}$ | 4 | $4\frac{1}{4}$ |

With 2 rivets in member deduct 2 rivets if b < b in table.
" 2 " " " " 1 " b > b " "
" 3 " " " " 3 " b < b " "
" 3 " " " " 2 " b > b " "

For $\frac{5}{8}$" rivets take b $\frac{1}{8}$" less than b for $\frac{3}{4}$".
" 1" " " b $\frac{1}{8}$" more than b for $\frac{7}{8}$".

## TABLE 31.

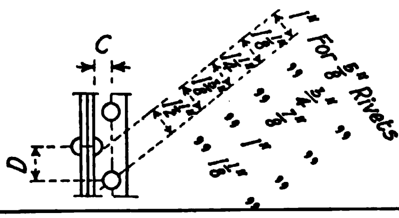
### Standards for Riveting.




**CLEARANCE FOR COVER PLATE RIVETING**

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| S | $\frac{1}{2}''$ | $1''$ | $1\frac{1}{2}''$ | $2''$ | $2\frac{1}{2}''$ | $3''$ | $3\frac{1}{2}''$ | $4''$ | $4\frac{1}{2}''$ | $5''$ | $5\frac{1}{2}''$ | $6''$ |
| W | $2\frac{1}{2}$ | $2\frac{5}{8}$ | $2\frac{3}{4}$ | $2\frac{3}{4}$ | $2\frac{7}{8}$ | $2\frac{7}{8}$ | 3 | $3\frac{1}{8}$ | $3\frac{1}{8}$ | $3\frac{1}{4}$ | $3\frac{1}{4}$ | $3\frac{3}{8}$ |
| $S_1$ | 0 | $\frac{1}{2}$ | 1 | $1\frac{1}{2}$ | 2 | $2\frac{1}{2}$ | | | | | | |
| W | $2\frac{1}{2}$ | $2\frac{1}{4}$ | $2\frac{1}{8}$ | 2 | $1\frac{1}{2}$ | 0 | | | | | | |

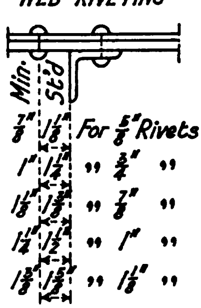
**MINIMUM STAGGER FOR RIVETS**



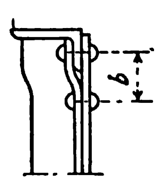

**Value of D in Inches.**

| C / Riv. | $1\frac{1}{8}''$ | $1\frac{3}{16}''$ | $1\frac{1}{4}''$ | $1\frac{5}{16}''$ | $1\frac{3}{8}''$ | $1\frac{7}{16}''$ | $1\frac{1}{2}''$ | $1\frac{9}{16}''$ | $1\frac{5}{8}''$ | $1\frac{11}{16}''$ | $1\frac{3}{4}''$ | $1\frac{13}{16}''$ | $1\frac{7}{8}''$ | $1\frac{15}{16}''$ | $2\frac{1}{16}''$ | $2\frac{3}{16}''$ | $2\frac{5}{16}''$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $\frac{5}{8}''$ | $\frac{15}{16}$ | $\frac{7}{8}$ | $\frac{13}{16}$ | $\frac{11}{16}$ | $\frac{1}{2}$ | $\frac{5}{16}$ | 0 | | | | | | | | | | |
| $\frac{3}{4}$ | $1\frac{1}{4}$ | $1\frac{3}{16}$ | $1\frac{1}{8}$ | $1\frac{1}{16}$ | $\frac{15}{16}$ | $\frac{7}{8}$ | $\frac{3}{4}$ | $\frac{9}{16}$ | $\frac{3}{8}$ | 0 | | | | | | | |
| $\frac{7}{8}$ | $1\frac{1}{2}$ | $1\frac{7}{16}$ | $1\frac{3}{8}$ | $1\frac{5}{16}$ | $1\frac{1}{4}$ | $1\frac{3}{16}$ | $1\frac{1}{8}$ | 1 | $\frac{15}{16}$ | $\frac{13}{16}$ | $\frac{5}{8}$ | $\frac{7}{16}$ | 0 | | | | |
| 1 | $1\frac{13}{16}$ | $1\frac{3}{4}$ | $1\frac{11}{16}$ | $1\frac{5}{8}$ | $1\frac{9}{16}$ | $1\frac{1}{2}$ | $1\frac{7}{16}$ | $1\frac{3}{8}$ | $1\frac{5}{16}$ | $1\frac{3}{16}$ | $1\frac{1}{8}$ | 1 | $\frac{7}{8}$ | $\frac{3}{4}$ | 0 | | |
| $1\frac{1}{8}$ | $2\frac{1}{16}$ | 2 | $1\frac{15}{16}$ | $1\frac{15}{16}$ | $1\frac{7}{8}$ | $1\frac{13}{16}$ | $1\frac{3}{4}$ | $1\frac{11}{16}$ | $1\frac{5}{8}$ | $1\frac{9}{16}$ | $1\frac{1}{2}$ | $1\frac{3}{8}$ | $1\frac{5}{16}$ | $1\frac{1}{4}$ | 1 | $\frac{11}{16}$ | 0 |

**CLEARANCE FOR WEB RIVETING**

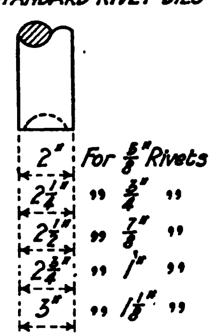



**RIVETS IN CRIMPED ANGLES**




Distance "b" should be $1\frac{1}{2}''$ plus thickness of chord angles, but never less than 2".

**STANDARD RIVET DIES**



## TABLE 32.

**AMERICAN BRIDGE COMPANY STANDARD.**

**Lengths of Field Rivets for Various Grips.**

**Dimensions in Inches.**


| Grip a. | Diameter. | | | | | Grip b. | Diameter. | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1/2 | 5/8 | 3/4 | 7/8 | 1 | | 1/2 | 5/8 | 3/4 | 7/8 | 1 |
| 1/2 | 1 1/2 | 1 3/4 | 1 7/8 | 2 | 2 1/8 | 1/2 | 1 1/8 | 1 1/4 | 1 1/4 | 1 3/8 | 1 3/8 |
| 5/8 | 1 5/8 | 1 7/8 | 2 | 2 1/8 | 2 1/4 | 5/8 | 1 1/4 | 1 3/8 | 1 3/8 | 1 1/2 | 1 1/2 |
| 3/4 | 1 3/4 | 2 | 2 1/8 | 2 1/4 | 2 3/8 | 3/4 | 1 3/8 | 1 1/2 | 1 1/2 | 1 5/8 | 1 5/8 |
| 7/8 | 1 7/8 | 2 1/8 | 2 1/4 | 2 3/8 | 2 1/2 | 7/8 | 1 1/2 | 1 5/8 | 1 5/8 | 1 3/4 | 1 3/4 |
| 1 | 2 | 2 1/4 | 2 3/8 | 2 1/2 | 2 5/8 | 1 | 1 5/8 | 1 3/4 | 1 3/4 | 1 7/8 | 1 7/8 |
| 1/8 | 2 1/8 | 2 3/8 | 2 1/2 | 2 5/8 | 2 3/4 | 1/8 | 1 3/4 | 1 7/8 | 1 7/8 | 2 | 2 |
| 1/4 | 2 1/4 | 2 1/2 | 2 5/8 | 2 3/4 | 2 7/8 | 1/4 | 1 7/8 | 2 | 2 | 2 1/8 | 2 1/8 |
| 3/8 | 2 3/8 | 2 5/8 | 2 3/4 | 2 7/8 | 3 | 3/8 | 2 | 2 1/8 | 2 1/8 | 2 1/4 | 2 1/4 |
| 1/2 | 2 5/8 | 2 7/8 | 3 | 3 1/8 | 3 1/4 | 1/2 | 2 1/8 | 2 1/4 | 2 3/8 | 2 3/8 | 2 1/2 |
| 5/8 | 2 3/4 | 3 | 3 1/8 | 3 1/4 | 3 3/8 | 5/8 | 2 1/4 | 2 3/8 | 2 1/2 | 2 1/2 | 2 5/8 |
| 3/4 | 3 | 3 1/4 | 3 3/8 | 3 1/2 | 3 5/8 | 3/4 | 2 1/2 | 2 5/8 | 2 3/4 | 2 3/4 | 2 7/8 |
| 7/8 | 3 1/8 | 3 3/8 | 3 1/2 | 3 5/8 | 3 3/4 | 7/8 | 2 5/8 | 2 3/4 | 2 7/8 | 2 7/8 | 3 |
| 2 | 3 1/4 | 3 1/2 | 3 5/8 | 3 3/4 | 3 7/8 | 2 | 2 3/4 | 2 7/8 | 3 | 3 | 3 1/8 |
| 1/8 | 3 3/8 | 3 5/8 | 3 3/4 | 3 7/8 | 4 | 1/8 | 2 7/8 | 3 | 3 1/8 | 3 1/8 | 3 1/4 |
| 1/4 | 3 1/2 | 3 3/4 | 3 7/8 | 4 | 4 1/8 | 1/4 | 3 | 3 1/8 | 3 1/4 | 3 1/4 | 3 3/8 |
| 3/8 | 3 5/8 | 3 7/8 | 4 | 4 1/8 | 4 1/4 | 3/8 | 3 1/8 | 3 1/4 | 3 3/8 | 3 3/8 | 3 1/2 |
| 1/2 | 3 3/4 | 4 | 4 1/8 | 4 1/4 | 4 3/8 | 1/2 | 3 1/4 | 3 3/8 | 3 1/2 | 3 1/2 | 3 5/8 |
| 5/8 | 3 7/8 | 4 1/8 | 4 1/4 | 4 3/8 | 4 1/2 | 5/8 | 3 3/8 | 3 1/2 | 3 5/8 | 3 5/8 | 3 3/4 |
| 3/4 | 4 | 4 1/4 | 4 3/8 | 4 1/2 | 4 5/8 | 3/4 | 3 1/2 | 3 5/8 | 3 3/4 | 3 3/4 | 3 7/8 |
| 7/8 | 4 1/8 | 4 3/8 | 4 1/2 | 4 5/8 | 4 3/4 | 7/8 | 3 5/8 | 3 3/4 | 3 7/8 | 3 7/8 | 4 |
| 3 | 4 3/8 | 4 5/8 | 4 3/4 | 4 7/8 | 5 | 3 | 3 7/8 | 4 | 4 | 4 1/8 | 4 1/4 |
| 1/8 | 4 1/2 | 4 3/4 | 4 7/8 | 5 | 5 1/8 | 1/8 | 4 | 4 1/8 | 4 1/8 | 4 1/4 | 4 3/8 |
| 1/4 | 4 5/8 | 4 7/8 | 5 | 5 1/8 | 5 1/4 | 1/4 | 4 1/8 | 4 1/4 | 4 1/4 | 4 3/8 | 4 1/2 |
| 3/8 | 4 3/4 | 5 | 5 1/8 | 5 1/4 | 5 3/8 | 3/8 | 4 1/4 | 4 3/8 | 4 3/8 | 4 1/2 | 4 5/8 |
| 1/2 | 4 7/8 | 5 1/8 | 5 1/4 | 5 3/8 | 5 1/2 | 1/2 | 4 3/8 | 4 1/2 | 4 1/2 | 4 5/8 | 4 3/4 |
| 5/8 | 5 | 5 1/4 | 5 3/8 | 5 1/2 | 5 5/8 | 5/8 | 4 1/2 | 4 5/8 | 4 5/8 | 4 3/4 | 4 7/8 |
| 3/4 | 5 1/8 | 5 3/8 | 5 1/2 | 5 5/8 | 5 3/4 | 3/4 | 4 5/8 | 4 3/4 | 4 3/4 | 4 7/8 | 5 |
| 7/8 | 5 1/4 | 5 1/2 | 5 5/8 | 5 3/4 | 5 7/8 | 7/8 | 4 3/4 | 4 7/8 | 4 7/8 | 5 | 5 1/8 |
| 4 | 5 3/8 | 5 5/8 | 5 3/4 | 5 7/8 | 6 | 4 | 4 7/8 | 5 | 5 | 5 1/8 | 5 1/4 |
| 1/8 | 5 5/8 | 5 7/8 | 6 | 6 1/8 | 6 1/8 | 1/8 | 5 1/8 | 5 1/4 | 5 1/4 | 5 3/8 | 5 3/8 |
| 1/4 | 5 3/4 | 6 | 6 1/8 | 6 1/4 | 6 3/8 | 1/4 | 5 1/4 | 5 3/8 | 5 3/8 | 5 1/2 | 5 1/2 |
| 3/8 | 6 | 6 1/4 | 6 1/4 | 6 3/8 | 6 1/2 | 3/8 | 5 3/8 | 5 1/2 | 5 1/2 | 5 5/8 | 5 3/4 |
| 1/2 | 6 1/8 | 6 3/8 | 6 1/2 | 6 5/8 | 6 5/8 | 1/2 | 5 5/8 | 5 3/4 | 5 3/4 | 5 3/4 | 5 7/8 |
| 5/8 | 6 1/4 | 6 1/2 | 6 5/8 | 6 3/4 | 6 7/8 | 5/8 | 5 3/4 | 5 7/8 | 5 7/8 | 5 7/8 | 6 |
| 3/4 | 6 3/8 | 6 5/8 | 6 3/4 | 6 7/8 | 7 | 3/4 | 5 7/8 | 6 | 6 | 6 | 6 1/8 |
| 7/8 | 6 1/2 | 6 3/4 | 6 7/8 | 7 | 7 1/8 | 7/8 | 6 | 6 1/8 | 6 1/8 | 6 1/8 | 6 1/4 |
| 5 | 6 5/8 | 6 7/8 | 7 | 7 1/8 | 7 1/4 | 5 | 6 1/8 | 6 1/4 | 6 1/4 | 6 1/4 | 6 3/8 |
| 1/8 | ...... | ...... | 7 1/8 | 7 1/4 | 7 3/8 | 1/8 | ...... | ...... | 6 3/8 | 6 3/8 | 6 1/2 |
| 1/4 | ...... | ...... | 7 1/4 | 7 3/8 | 7 1/2 | 1/4 | ...... | ...... | 6 1/2 | 6 1/2 | 6 5/8 |
| 3/8 | ...... | ...... | 7 3/8 | 7 1/2 | 7 5/8 | 3/8 | ...... | ...... | 6 5/8 | 6 5/8 | 6 3/4 |
| 1/2 | ...... | ...... | 7 5/8 | 7 3/4 | 7 7/8 | 1/2 | ...... | ...... | 6 7/8 | 6 7/8 | 7 |
| 5/8 | ...... | ...... | 7 3/4 | 7 7/8 | 8 | 5/8 | ...... | ...... | 7 | 7 | 7 1/8 |
| 3/4 | ...... | ...... | 7 7/8 | 8 | 8 1/8 | 3/4 | ...... | ...... | 7 1/8 | 7 1/8 | 7 1/4 |
| 7/8 | ...... | ...... | 8 | 8 1/8 | 8 1/4 | 7/8 | ...... | ...... | 7 1/4 | 7 1/4 | 7 3/8 |

## TABLE 33

### Shearing and Bearing Value of Rivets

**Values above or to right of upper zigzag lines are greater than double shear.**
**Values below or to left of lower zigzag lines are less than single shear.**

| Rivet Diam., In. | Rivet Area, Sq. In. | Single Shear at 6 000 Pounds | Bearing Value for Different Thicknesses of Plate at 12 000 Lbs. Per Square Inch. | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1/4" | 5/16" | 3/8" | 7/16" | 1/2" | 9/16" | 5/8" | 11/16" | 3/4" | 13/16" | 7/8" | 15/16" | 1" |
| 1/2 | .196 | 1 180 | 1 500 | 1 880 | 2 250 | 2 630 | 3 000 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 5/8 | .307 | 1 840 | 1 880 | 2 340 | 2 810 | 3 280 | 3 750 | 4 220 | 4 690 | ..... | ..... | ..... | ..... | ..... | ..... |
| 3/4 | .442 | 2 650 | 2 250 | 2 810 | 3 380 | 3 940 | 4 500 | 5 060 | 5 630 | 6 190 | 6 750 | ..... | ..... | ..... | ..... |
| 7/8 | .601 | 3 610 | 2 630 | 3 280 | 3 940 | 4 590 | 5 250 | 5 910 | 6 560 | 7 220 | 7 880 | 8 530 | 9 190 | 9 840 | ..... |
| 1 | .785 | 4 710 | 3 000 | 3 750 | 4 500 | 5 260 | 6 000 | 6 750 | 7 500 | 8 250 | 9 000 | 9 750 | 10 500 | 11 250 | 12 000 |

| Rivet Diam., In. | Rivet Area, Sq. In. | Single Shear at 7 500 Pounds | Bearing Value for Different Thicknesses of Plate at 15 000 Lbs. Per Square Inch | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1/4" | 5/16" | 3/8" | 7/16" | 1/2" | 9/16" | 5/8" | 11/16" | 3/4" | 13/16" | 7/8" | 15/16" | 1" |
| 1/2 | .196 | 1 470 | 1 880 | 2 340 | 2 810 | 3 280 | 3 750 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 5/8 | .307 | 2 300 | 2 340 | 2 930 | 3 520 | 4 100 | 4 690 | 5 270 | 5 860 | ..... | ..... | ..... | ..... | ..... | ..... |
| 3/4 | .442 | 3 310 | 2 810 | 3 520 | 4 220 | 4 920 | 5 630 | 6 330 | 7 030 | 7 730 | 8 440 | ..... | ..... | ..... | ..... |
| 7/8 | .601 | 4 510 | 3 280 | 4 100 | 4 920 | 5 740 | 6 560 | 7 380 | 8 200 | 9 020 | 9 840 | 10 660 | 11 480 | 12 300 | ..... |
| 1 | .785 | 5 890 | 3 750 | 4 690 | 5 630 | 6 560 | 7 500 | 8 440 | 9 380 | 10 310 | 11 250 | 12 190 | 13 130 | 14 060 | 15 000 |

| Rivet Diam., In. | Rivet Area, Sq. In. | Single Shear at 10 000 Pounds | Bearing Value for Different Thicknesses of Plate at 20 000 Lbs. Per Square Inch | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1/4" | 5/16" | 3/8" | 7/16" | 1/2" | 9/16" | 5/8" | 11/16" | 3/4" | 13/16" | 7/8" | 15/16" | 1" |
| 1/2 | .196 | 1 960 | 2 500 | 3 130 | 3 750 | 4 380 | 5 000 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 5/8 | .307 | 3 070 | 3 130 | 3 910 | 4 690 | 5 470 | 6 250 | 7 030 | 7 810 | ..... | ..... | ..... | ..... | ..... | ..... |
| 3/4 | .442 | 4 420 | 3 750 | 4 690 | 5 630 | 6 560 | 7 500 | 8 440 | 9 380 | 10 310 | 11 250 | ..... | ..... | ..... | ..... |
| 7/8 | .601 | 6 010 | 4 380 | 5 470 | 6 560 | 7 660 | 8 750 | 9 840 | 10 940 | 12 030 | 13 130 | 14 220 | 15 310 | 16 410 | ..... |
| 1 | .785 | 7 850 | 5 000 | 6 250 | 7 500 | 8 750 | 10 000 | 11 250 | 12 500 | 13 750 | 15 000 | 16 250 | 17 500 | 18 750 | 20 000 |

| Rivet Diam., In. | Rivet Area, Sq. In. | Single Shear at 11 000 Pounds | Bearing Value for Different Thicknesses of Plate at 22 000 Lbs. Per Square Inch | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1/4" | 5/16" | 3/8" | 7/16" | 1/2" | 9/16" | 5/8" | 11/16" | 3/4" | 13/16" | 7/8" | 15/16" | 1" |
| 1/2 | .196 | 2 160 | 2 750 | 3 440 | 4 130 | 4 810 | 5 500 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 5/8 | .307 | 3 370 | 3 440 | 4 300 | 5 160 | 6 020 | 6 880 | 7 730 | 8 590 | ..... | ..... | ..... | ..... | ..... | ..... |
| 3/4 | .442 | 4 860 | 4 130 | 5 160 | 6 190 | 7 220 | 8 250 | 9 280 | 10 310 | 11 340 | 12 380 | ..... | ..... | ..... | ..... |
| 7/8 | .601 | 6 610 | 4 810 | 6 020 | 7 220 | 8 420 | 9 630 | 10 830 | 12 030 | 13 230 | 14 440 | 15 640 | 16 840 | 18 050 | ..... |
| 1 | .785 | 8 640 | 5 500 | 6 880 | 8 250 | 9 630 | 11 000 | 12 380 | 13 750 | 15 130 | 16 500 | 17 880 | 19 250 | 20 630 | 22 000 |

| Rivet Diam., In. | Rivet Area, Sq. In. | Single Shear at 12 000 Pounds | Bearing Value for Different Thicknesses of Plate at 24 000 Lbs. Per Square Inch | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | 1/4" | 5/16" | 3/8" | 7/16" | 1/2" | 9/16" | 5/8" | 11/16" | 3/4" | 13/16" | 7/8" | 15/16" | 1" |
| 1/2 | .196 | 2 360 | 3 000 | 3 750 | 4 500 | 5 250 | 6 000 | ..... | ..... | ..... | ..... | ..... | ..... | ..... | ..... |
| 5/8 | .307 | 3 680 | 3 750 | 4 690 | 5 630 | 6 560 | 7 500 | 8 440 | 9 380 | ..... | ..... | ..... | ..... | ..... | ..... |
| 3/4 | .442 | 5 300 | 4 500 | 5 630 | 6 750 | 7 880 | 9 000 | 10 130 | 11 250 | 12 380 | 13 500 | ..... | ..... | ..... | ..... |
| 7/8 | .601 | 7 220 | 5 250 | 6 560 | 7 880 | 9 190 | 10 500 | 11 810 | 13 130 | 14 440 | 15 750 | 17 060 | 18 380 | ..... | ..... |
| 1 | .785 | 9 420 | 6 000 | 7 500 | 9 000 | 10 500 | 12 000 | 13 500 | 15 000 | 16 500 | 18 000 | 19 500 | 21 000 | 22 500 | 24 000 |

## TABLE 34.

### Multiplication Table for Rivet Spacing

| Spaces | Pitch of Rivets in Inches | | | | | | | | | | | | | | | Spaces |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 1⅛ | 1¼ | 1⅜ | 1½ | 1⅝ | 1¾ | 1⅞ | 2 | 2⅛ | 2¼ | 2⅜ | 2½ | 2⅝ | 2¾ | 2⅞ | |
| 1 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1 |
| 2 | – 2¼ | – 2½ | – 2¾ | – 3 | – 3¼ | – 3½ | – 3¾ | – 4 | – 4¼ | – 4½ | – 4¾ | – 5 | – 5¼ | – 5½ | – 5¾ | 2 |
| 3 | – 3⅜ | – 3¾ | – 4⅛ | – 4½ | – 4⅞ | – 5¼ | – 5⅝ | – 6 | – 6⅜ | – 6¾ | – 7⅛ | – 7½ | – 7⅞ | – 8¼ | – 8⅝ | 3 |
| 4 | – 4½ | – 5 | – 5½ | – 6 | – 6½ | – 7 | – 7½ | – 8 | – 8½ | – 9 | – 9½ | –10 | –10½ | –11 | –11½ | 4 |
| 5 | – 5⅝ | – 6¼ | – 6⅞ | – 7½ | – 8⅛ | – 8¾ | – 9⅜ | –10 | –10⅝ | –11¼ | –11⅞ | 1– 0½ | 1– 1⅛ | 1– 1¾ | 1– 2⅜ | 5 |
| 6 | – 6¾ | – 7½ | – 8¼ | – 9 | – 9¾ | –10½ | –11¼ | 1– 0 | 1– 0¾ | 1– 1½ | 1– 2¼ | 1– 3 | 1– 3¾ | 1– 4½ | 1– 5¼ | 6 |
| 7 | – 7⅞ | – 8¾ | – 9⅝ | –10½ | –11⅜ | 1– 0¼ | 1– 1⅛ | 1– 2 | 1– 2⅞ | 1– 3¾ | 1– 4⅝ | 1– 5½ | 1– 6⅜ | 1– 7¼ | 1– 8⅛ | 7 |
| 8 | – 9 | –10 | –11 | 1– 0 | 1– 1 | 1– 2 | 1– 3 | 1– 4 | 1– 5 | 1– 6 | 1– 7 | 1– 8 | 1– 9 | 1–10 | 1–11 | 8 |
| 9 | –10⅛ | –11¼ | 1– 0⅜ | 1– 1½ | 1– 2⅝ | 1– 3¾ | 1– 4⅞ | 1– 6 | 1– 7⅛ | 1– 8¼ | 1– 9⅜ | 1–10½ | 1–11⅝ | 2– 0¾ | 2– 1⅞ | 9 |
| 10 | –11¼ | 1– 0½ | 1– 1¾ | 1– 3 | 1– 4¼ | 1– 5½ | 1– 6¾ | 1– 8 | 1– 9¼ | 1–10½ | 1–11¾ | 2– 1 | 2– 2¼ | 2– 3½ | 2– 4¾ | 10 |
| 11 | 1– 0⅜ | 1– 1¾ | 1– 3⅛ | 1– 4½ | 1– 5⅞ | 1– 7¼ | 1– 8⅝ | 1–10 | 1–11⅜ | 2– 0¾ | 2– 2⅛ | 2– 3½ | 2– 4⅞ | 2– 6¼ | 2– 7⅝ | 11 |
| 12 | 1– 1½ | 1– 3 | 1– 4½ | 1– 6 | 1– 7½ | 1– 9 | 1–10½ | 2– 0 | 2– 1½ | 2– 3 | 2– 4½ | 2– 6 | 2– 7½ | 2– 9 | 2–10½ | 12 |
| 13 | 1– 2⅝ | 1– 4¼ | 1– 5⅞ | 1– 7½ | 1– 9⅛ | 1–10¾ | 2– 0⅜ | 2– 2 | 2– 3⅝ | 2– 5¼ | 2– 6⅞ | 2– 8½ | 2–10⅛ | 2–11¾ | 3– 1⅜ | 13 |
| 14 | 1– 3¾ | 1– 5½ | 1– 7¼ | 1– 9 | 1–10¾ | 2– 0½ | 2– 2¼ | 2– 4 | 2– 5¾ | 2– 7½ | 2– 9¼ | 2–11 | 3– 0¾ | 3– 2½ | 3– 4¼ | 14 |
| 15 | 1– 4⅞ | 1– 6¾ | 1– 8⅝ | 1–10½ | 2– 0⅜ | 2– 2¼ | 2– 4⅛ | 2– 6 | 2– 7⅞ | 2– 9¾ | 2–11⅝ | 3– 1½ | 3– 3⅜ | 3– 5¼ | 3– 7⅛ | 15 |
| 16 | 1– 6 | 1– 8 | 1–10 | 2– 0 | 2– 2 | 2– 4 | 2– 6 | 2– 8 | 2–10 | 3– 0 | 3– 2 | 3– 4 | 3– 6 | 3– 8 | 3–10 | 16 |
| 17 | 1– 7⅛ | 1– 9¼ | 1–11⅜ | 2– 1½ | 2– 3⅝ | 2– 5¾ | 2– 7⅞ | 2–10 | 3– 0⅛ | 3– 2¼ | 3– 4⅜ | 3– 6½ | 3– 8⅝ | 3–10¾ | 4– 0⅞ | 17 |
| 18 | 1– 8¼ | 1–10½ | 2– 0¾ | 2– 3 | 2– 5¼ | 2– 7½ | 2– 9¾ | 3– 0 | 3– 2¼ | 3– 4½ | 3– 6¾ | 3– 9 | 3–11¼ | 4– 1½ | 4– 3¾ | 18 |
| 19 | 1– 9⅜ | 1–11¾ | 2– 2⅛ | 2– 4½ | 2– 6⅞ | 2– 9¼ | 2–11⅝ | 3– 2 | 3– 4⅜ | 3– 6¾ | 3– 9⅛ | 3–11½ | 4– 1⅞ | 4– 4¼ | 4– 6⅝ | 19 |
| 20 | 1–10½ | 2– 1 | 2– 3½ | 2– 6 | 2– 8½ | 2–11 | 3– 1½ | 3– 4 | 3– 6½ | 3– 9 | 3–11½ | 4– 2 | 4– 4½ | 4– 7 | 4– 9½ | 20 |
| 21 | 1–11⅝ | 2– 2¼ | 2– 4⅞ | 2– 7½ | 2–10⅛ | 3– 0¾ | 3– 3⅜ | 3– 6 | 3– 8⅝ | 3–11¼ | 4– 1⅞ | 4– 4½ | 4– 7⅛ | 4– 9¾ | 5– 0⅜ | 21 |
| 22 | 2– 0¾ | 2– 3½ | 2– 6¼ | 2– 9 | 2–11¾ | 3– 2½ | 3– 5¼ | 3– 8 | 3–10¾ | 4– 1½ | 4– 4¼ | 4– 7 | 4– 9¾ | 5– 0½ | 5– 3¼ | 22 |
| 23 | 2– 1⅞ | 2– 4¾ | 2– 7⅝ | 2–10½ | 3– 1⅜ | 3– 4¼ | 3– 7⅛ | 3–10 | 4– 0⅞ | 4– 3¾ | 4– 6⅝ | 4– 9½ | 5– 0⅜ | 5– 3¼ | 5– 6⅛ | 23 |
| 24 | 2– 3 | 2– 6 | 2– 9 | 3– 0 | 3– 3 | 3– 6 | 3– 9 | 4– 0 | 4– 3 | 4– 6 | 4– 9 | 5– 0 | 5– 3 | 5– 6 | 5– 9 | 24 |
| 25 | 2– 4⅛ | 2– 7¼ | 2–10⅜ | 3– 1½ | 3– 4⅝ | 3– 7¾ | 3–10⅞ | 4– 2 | 4– 5⅛ | 4– 8¼ | 4–11⅜ | 5– 2½ | 5– 5⅝ | 5– 8¾ | 5–11⅞ | 25 |
| 26 | 2– 5¼ | 2– 8½ | 2–11¾ | 3– 3 | 3– 6¼ | 3– 9½ | 4– 0¾ | 4– 4 | 4– 7¼ | 4–10½ | 5– 1¾ | 5– 5 | 5– 8¼ | 5–11½ | 6– 2¾ | 26 |
| 27 | 2– 6⅜ | 2– 9¾ | 3– 1⅛ | 3– 4½ | 3– 7⅞ | 3–11¼ | 4– 2⅝ | 4– 6 | 4– 9⅜ | 5– 0¾ | 5– 4⅛ | 5– 7½ | 5–10⅞ | 6– 2¼ | 6– 5⅝ | 27 |
| 28 | 2– 7½ | 2–11 | 3– 2½ | 3– 6 | 3– 9½ | 4– 1 | 4– 4½ | 4– 8 | 4–11½ | 5– 3 | 5– 6½ | 5–10 | 6– 1½ | 6– 5 | 6– 8½ | 28 |
| 29 | 2– 8⅝ | 3– 0¼ | 3– 3⅞ | 3– 7½ | 3–11⅛ | 4– 2¾ | 4– 6⅜ | 4–10 | 5– 1⅝ | 5– 5¼ | 5– 8⅞ | 6– 0½ | 6– 4⅛ | 6– 7¾ | 6–11⅜ | 29 |
| 30 | 2– 9¾ | 3– 1½ | 3– 5¼ | 3– 9 | 4– 0¾ | 4– 4½ | 4– 8¼ | 5– 0 | 5– 3¾ | 5– 7½ | 5–11¼ | 6– 3 | 6– 6¾ | 6–10½ | 7– 2¼ | 30 |
| Spaces | 1⅛ | 1¼ | 1⅜ | 1½ | 1⅝ | 1¾ | 1⅞ | 2 | 2⅛ | 2¼ | 2⅜ | 2½ | 2⅝ | 2¾ | 2⅞ | Spaces |
| | Pitch of Rivets in Inches | | | | | | | | | | | | | | | |

TABLE 34.—*Continued*

MULTIPLICATION TABLE FOR RIVET SPACING

| Spaces | Pitch of Rivets in Inches | | | | | | | | | | | | | | | Spaces |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 3 | 3⅛ | 3¼ | 3⅜ | 3½ | 3¾ | 4 | 4¼ | 4½ | 4¾ | 5 | 5¼ | 5½ | 5¾ | 6 | |
| 1 | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | .... | 1 |
| 2 | -6 | - 6¼ | - 6½ | - 6¾ | - 7 | - 7½ | -8 | - 8½ | - 9 | - 9½ | -10 | -10½ | -11 | -11½ | 1-0 | 2 |
| 3 | -9 | - 9⅜ | - 9¾ | -10⅛ | -10½ | -11¼ | 1-0 | 1- 0¾ | 1- 1½ | 1- 2¼ | 1- 3 | 1- 3¾ | 1- 4½ | 1- 5¼ | 1-6 | 3 |
| 4 | 1-0 | 1- 0½ | 1- 1 | 1- 1½ | 1- 2 | 1- 3 | 1-4 | 1- 5 | 1- 6 | 1- 7 | 1- 8 | 1- 9 | 1-10 | 1-11 | 2-0 | 4 |
| 5 | 1-3 | 1- 3⅝ | 1- 4¼ | 1- 4⅞ | 1- 5½ | 1- 6¾ | 1-8 | 1- 9¼ | 1-10½ | 1-11¾ | 2- 1 | 2- 2¼ | 2- 3½ | 2- 4¾ | 2-6 | 5 |
| 6 | 1-6 | 1- 6¾ | 1- 7½ | 1- 8¼ | 1- 9 | 1-10½ | 2-0 | 2- 1½ | 2- 3 | 2- 4½ | 2- 6 | 2- 7½ | 2- 9 | 2-10½ | 3-0 | 6 |
| 7 | 1-9 | 1- 9⅞ | 1-10¾ | 1-11⅝ | 2- 0½ | 2- 2¼ | 2-4 | 2- 5¾ | 2- 7½ | 2- 9¼ | 2-11 | 3- 0¾ | 3- 2½ | 3- 4¼ | 3-6 | 7 |
| 8 | 2-0 | 2- 1 | 2- 2 | 2- 3 | 2- 4 | 2- 6 | 2-8 | 2-10 | 3- 0 | 3- 2 | 3- 4 | 3- 6 | 3- 8 | 3-10 | 4-0 | 8 |
| 9 | 2-3 | 2- 4⅛ | 2- 5¼ | 2- 6⅜ | 2- 7½ | 2- 9¾ | 3-0 | 3- 2¼ | 3- 4½ | 3- 6¾ | 3- 9 | 3-11¼ | 4- 1½ | 4- 3¾ | 4-6 | 9 |
| 10 | 2-6 | 2- 7¼ | 2- 8½ | 2- 9¾ | 2-11 | 3- 1½ | 3-4 | 3- 6½ | 3- 9 | 3-11½ | 4- 2 | 4- 4½ | 4- 7 | 4- 9½ | 5-0 | 10 |
| 11 | 2-9 | 2-10⅜ | 2-11¾ | 3- 1⅛ | 3- 2½ | 3- 5¼ | 3-8 | 3-10¾ | 4- 1½ | 4- 4¼ | 4- 7 | 4- 9¾ | 5- 0½ | 5- 3¼ | 5-6 | 11 |
| 12 | 3-0 | 3- 1½ | 3- 3 | 3- 4½ | 3- 6 | 3- 9 | 4-0 | 4- 3 | 4- 6 | 4- 9 | 5- 0 | 5- 3 | 5- 6 | 5- 9 | 6-0 | 12 |
| 13 | 3-3 | 3- 4⅝ | 3- 6¼ | 3- 7⅞ | 3- 9½ | 4- 0¾ | 4-4 | 4- 7¼ | 4-10½ | 5- 1¾ | 5- 5 | 5- 8¼ | 5-11½ | 6- 2¾ | 6-6 | 13 |
| 14 | 3-6 | 3- 7¾ | 3- 9½ | 3-11¼ | 4- 1 | 4- 4½ | 4-8 | 4-11½ | 5- 3 | 5- 6½ | 5-10 | 6- 1½ | 6- 5 | 6- 8½ | 7-0 | 14 |
| 15 | 3-9 | 3-10⅞ | 4- 0¾ | 4- 2⅝ | 4- 4½ | 4- 8¼ | 5-0 | 5- 3¾ | 5- 7½ | 5-11¼ | 6- 3 | 6- 6¾ | 6-10½ | 7- 2¼ | 7-6 | 15 |
| 16 | 4-0 | 4- 2 | 4- 4 | 4- 6 | 4- 8 | 5- 0 | 5-4 | 5- 8 | 6- 0 | 6- 4 | 6- 8 | 7- 0 | 7- 4 | 7- 8 | 8-0 | 16 |
| 17 | 4-3 | 4- 5⅛ | 4- 7¼ | 4- 9⅜ | 4-11½ | 5- 3¾ | 5-8 | 6- 0¼ | 6- 4½ | 6- 8¾ | 7- 1 | 7- 5¼ | 7- 9½ | 8- 1¾ | 8-6 | 17 |
| 18 | 4-6 | 4- 8¼ | 4-10½ | 5- 0¾ | 5- 3 | 5- 7½ | 6-0 | 6- 4½ | 6- 9 | 7- 1½ | 7- 6 | 7-10½ | 8- 3 | 8- 7½ | 9-0 | 18 |
| 19 | 4-9 | 4-11⅜ | 5- 1¾ | 5- 4⅛ | 5- 6½ | 5-11¼ | 6-4 | 6- 8¾ | 7- 1½ | 7- 6¼ | 7-11 | 8- 3¾ | 8- 8½ | 9- 1¼ | 9-6 | 19 |
| 20 | 5-0 | 5- 2½ | 5- 5 | 5- 7½ | 5-10 | 6- 3 | 6-8 | 7- 1 | 7- 6 | 7-11 | 8- 4 | 8- 9 | 9- 2 | 9- 7 | 10-0 | 20 |
| 21 | 5-3 | 5- 5⅝ | 5- 8¼ | 5-10⅞ | 6- 1½ | 6- 6¾ | 7-0 | 7- 5¼ | 7-10½ | 8- 3¾ | 8- 9 | 9- 2¼ | 9- 7½ | 10- 0¾ | 10-6 | 21 |
| 22 | 5-6 | 5- 8¾ | 5-11½ | 6- 2¼ | 6- 5 | 6-10½ | 7-4 | 7- 9½ | 8- 3 | 8- 8½ | 9- 2 | 9- 7½ | 10- 1 | 10- 6½ | 11-0 | 22 |
| 23 | 5-9 | 5-11⅞ | 6- 2¾ | 6- 5⅝ | 6- 8½ | 7- 2¼ | 7-8 | 8- 1¾ | 8- 7½ | 9- 1¼ | 9- 7 | 10- 0¾ | 10- 6½ | 11- 0¼ | 11-6 | 23 |
| 24 | 6-0 | 6- 3 | 6- 6 | 6- 9 | 7- 0 | 7- 6 | 8-0 | 8- 6 | 9- 0 | 9- 6 | 10- 0 | 10- 6 | 11- 0 | 11- 6 | 12-0 | 24 |
| 25 | 6-3 | 6- 6⅛ | 6- 9¼ | 7- 0⅜ | 7- 3½ | 7- 9¾ | 8-4 | 8-10¼ | 9- 4½ | 9-10¾ | 10- 5 | 10-11¼ | 11- 5½ | 11-11¾ | 12-6 | 25 |
| 26 | 6-6 | 6- 9¼ | 7- 0½ | 7- 3¾ | 7- 7 | 8- 1½ | 8-8 | 9- 2½ | 9- 9 | 10- 3½ | 10-10 | 11- 4½ | 11-11 | 12- 5½ | 13-0 | 26 |
| 27 | 6-9 | 7- 0⅜ | 7- 3¾ | 7- 7⅛ | 7-10½ | 8- 5¼ | 9-0 | 9- 6¾ | 10- 1½ | 10- 8¼ | 11- 3 | 11- 9¾ | 12- 4½ | 12-11¼ | 13-6 | 27 |
| 28 | 7-0 | 7- 3½ | 7- 7 | 7-10½ | 8- 2 | 8- 9 | 9-4 | 9-11 | 10- 6 | 11- 1 | 11- 8 | 12- 3 | 12-10 | 13- 5 | 14-0 | 28 |
| 29 | 7-3 | 7- 6⅝ | 7-10¼ | 8- 1⅞ | 8- 5½ | 9- 0¾ | 9-8 | 10- 3¼ | 10-10½ | 11- 5¾ | 12- 1 | 12- 8¼ | 13- 3½ | 13-10¾ | 14-6 | 29 |
| 30 | 7-6 | 7- 9¾ | 8- 1½ | 8- 5¼ | 8- 9 | 9- 4½ | 10-0 | 10- 7½ | 11- 3 | 11-10½ | 12- 6 | 13- 1½ | 13- 9 | 14- 4½ | 15-0 | 30 |
| Spaces | 3 | 3⅛ | 3¼ | 3⅜ | 3½ | 3¾ | 4 | 4¼ | 4½ | 4¾ | 5 | 5¼ | 5½ | 5¾ | 6 | Spaces |
| | Pitch of Rivets in Inches | | | | | | | | | | | | | | | |

## TABLE 35.

### Areas to be Deducted for Rivet Holes, Maximum Rivets, and Rivet Spacing.

**Areas in Square Inches, to be Deducted from Riveted Plates or Shapes to Obtain Net Areas.**

| Thickness of Plates. | Diameter of Hole in Inches (Diam. of Rivet + 1/8″). | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Inches. | 1/4 | 5/16 | 3/8 | 7/16 | 1/2 | 9/16 | 5/8 | 11/16 | 3/4 | 13/16 | 7/8 | 15/16 | 1 | 1 1/16 | 1 1/8 | 1 3/16 | 1 1/4 |
| 1/4 | .06 | .08 | .09 | .11 | .13 | .14 | .16 | .17 | .19 | .20 | .22 | .23 | .25 | .27 | .28 | .30 | .31 |
| 5/16 | .08 | .10 | .12 | .14 | .16 | .18 | .20 | .21 | .23 | .25 | .27 | .29 | .31 | .33 | .35 | .37 | .39 |
| 3/8 | .09 | .12 | .14 | .16 | .19 | .21 | .23 | .26 | .28 | .30 | .33 | .35 | .38 | .40 | .42 | .45 | .47 |
| 7/16 | .11 | .14 | .16 | .19 | .22 | .25 | .27 | .30 | .33 | .36 | .38 | .41 | .44 | .46 | .49 | .52 | .55 |
| 1/2 | .13 | .16 | .19 | .22 | .25 | .28 | .31 | .34 | .38 | .41 | .44 | .47 | .50 | .53 | .56 | .59 | .63 |
| 9/16 | .14 | .18 | .21 | .25 | .28 | .32 | .35 | .39 | .42 | .46 | .49 | .53 | .56 | .60 | .63 | .67 | .70 |
| 5/8 | .16 | .20 | .23 | .27 | .31 | .35 | .39 | .43 | .47 | .51 | .55 | .59 | .63 | .66 | .70 | .74 | .78 |
| 11/16 | .17 | .21 | .26 | .30 | .34 | .39 | .43 | .47 | .52 | .56 | .60 | .64 | .69 | .73 | .77 | .82 | .86 |
| 3/4 | .19 | .23 | .28 | .33 | .38 | .42 | .47 | .52 | .56 | .61 | .66 | .70 | .75 | .80 | .84 | .89 | .94 |
| 13/16 | .20 | .25 | .30 | .36 | .41 | .46 | .51 | .56 | .61 | .66 | .71 | .76 | .81 | .86 | .91 | .96 | 1.02 |
| 7/8 | .22 | .27 | .33 | .38 | .44 | .49 | .55 | .60 | .66 | .71 | .77 | .82 | .88 | .93 | .98 | 1.04 | 1.09 |
| 15/16 | .23 | .29 | .35 | .41 | .47 | .53 | .59 | .64 | .70 | .76 | .82 | .88 | .94 | 1.00 | 1.05 | 1.11 | 1.17 |
| 1 | .25 | .31 | .38 | .44 | .50 | .56 | .63 | .69 | .75 | .81 | .88 | .94 | 1.00 | 1.06 | 1.13 | 1.19 | 1.25 |
| 1 1/16 | .27 | .33 | .40 | .46 | .53 | .60 | .66 | .73 | .80 | .86 | .93 | 1.00 | 1.06 | 1.13 | 1.20 | 1.26 | 1.33 |
| 1 1/8 | .28 | .35 | .42 | .49 | .56 | .63 | .70 | .77 | .84 | .91 | .98 | 1.05 | 1.13 | 1.20 | 1.27 | 1.34 | 1.41 |
| 1 3/16 | .30 | .37 | .45 | .52 | .59 | .67 | .74 | .82 | .89 | .96 | 1.04 | 1.11 | 1.19 | 1.26 | 1.34 | 1.41 | 1.48 |
| 1 1/4 | .31 | .39 | .47 | .55 | .63 | .70 | .78 | .86 | .94 | 1.02 | 1.09 | 1.17 | 1.25 | 1.33 | 1.41 | 1.48 | 1.56 |
| 1 5/16 | .33 | .41 | .49 | .57 | .66 | .74 | .82 | .90 | .98 | 1.07 | 1.15 | 1.23 | 1.31 | 1.39 | 1.48 | 1.56 | 1.64 |
| 1 3/8 | .34 | .43 | .52 | .60 | .69 | .77 | .86 | .95 | 1.03 | 1.12 | 1.20 | 1.29 | 1.38 | 1.46 | 1.55 | 1.63 | 1.72 |
| 1 7/16 | .36 | .45 | .54 | .63 | .72 | .81 | .90 | .99 | 1.08 | 1.17 | 1.26 | 1.35 | 1.44 | 1.53 | 1.62 | 1.71 | 1.80 |
| 1 1/2 | .38 | .47 | .56 | .66 | .75 | .84 | .94 | 1.03 | 1.13 | 1.22 | 1.31 | 1.41 | 1.50 | 1.59 | 1.69 | 1.78 | 1.88 |
| 1 9/16 | .39 | .49 | .59 | .68 | .78 | .88 | .98 | 1.07 | 1.17 | 1.27 | 1.37 | 1.46 | 1.56 | 1.66 | 1.76 | 1.86 | 1.95 |
| 1 5/8 | .41 | .51 | .61 | .71 | .81 | .91 | 1.02 | 1.12 | 1.22 | 1.32 | 1.42 | 1.52 | 1.63 | 1.73 | 1.83 | 1.93 | 2.03 |
| 1 11/16 | .42 | .53 | .63 | .74 | .84 | .95 | 1.05 | 1.16 | 1.27 | 1.37 | 1.47 | 1.58 | 1.69 | 1.79 | 1.90 | 2.00 | 2.11 |
| 1 3/4 | .44 | .55 | .66 | .77 | .88 | .98 | 1.09 | 1.20 | 1.31 | 1.42 | 1.53 | 1.64 | 1.75 | 1.86 | 1.97 | 2.08 | 2.19 |
| 1 13/16 | .45 | .57 | .68 | .79 | .91 | 1.02 | 1.13 | 1.25 | 1.36 | 1.47 | 1.59 | 1.70 | 1.81 | 1.93 | 2.04 | 2.15 | 2.27 |
| 1 7/8 | .47 | .59 | .70 | .82 | .94 | 1.05 | 1.17 | 1.29 | 1.41 | 1.52 | 1.64 | 1.76 | 1.88 | 1.99 | 2.11 | 2.23 | 2.34 |
| 1 15/16 | .48 | .61 | .73 | .85 | .97 | 1.09 | 1.21 | 1.33 | 1.45 | 1.57 | 1.70 | 1.82 | 1.94 | 2.06 | 2.18 | 2.30 | 2.42 |
| 2 | .50 | .63 | .75 | .88 | 1.00 | 1.13 | 1.25 | 1.38 | 1.50 | 1.63 | 1.75 | 1.88 | 2.00 | 2.13 | 2.25 | 2.38 | 2.50 |

**Maximum Rivet in Leg of Angles or Flange of Beams and Channels.**

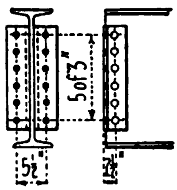
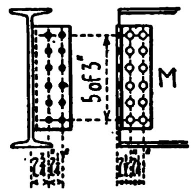
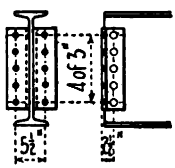
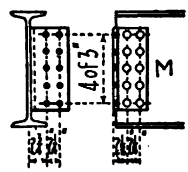
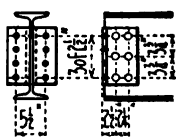
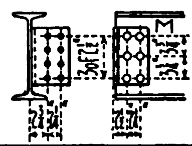
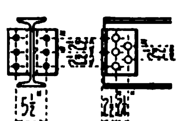
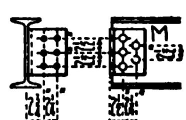
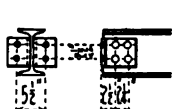
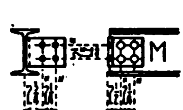
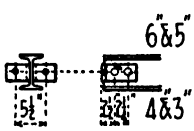
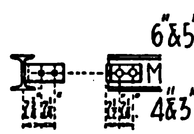
| | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Leg of Angle | 3/4 | 1 | 1 1/4 | 1 3/8 | 1 1/2 | 1 3/4 | 2 | 2 1/2 | 3 | 3 1/2 | 4 | 5 | 6 | 7 | 8 |
| Max. Rivet | 1/4 | 1/4 | 3/8 | 3/8 | 3/8 | 1/2 | 5/8 | 3/4 | 7/8 | 7/8 | 7/8 | 7/8 | 7/8 | 1 | 1 1/8 |
| Depth of Beam | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 12 | 15 | 18 | 20 | 24 | | |
| Max. Rivet | 3/8 | 1/2 | 1/2 | 5/8 | 5/8 | 3/4 | 3/4 | 3/4 | 3/4 | 3/4 | 7/8 | 7/8 | 7/8 | | |
| Depth of Channel | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 | 12 | 15 | | | | | |
| Max. Rivet | 1/2 | 1/2 | 1/2 | 5/8 | 5/8 | 3/4 | 3/4 | 3/4 | 7/8 | 7/8 | | | | | |

**Rivet Spacing in Inches.**

| Size of Rivet. | Minimum Pitch. | | Max. Pitch in Line of Stress. | | | Min. Edge Dist. | | Max. Edge Dist. |
|---|---|---|---|---|---|---|---|---|
| | Allowed. | Preferred. | At Ends of Comp. Mem. | Bridges. | Bld'gs. | Sheared. | Rolled. | |
| 1/2″ | 1 1/2 | 1 3/4 | 2 | 4 / 16 x thickness of thinnest outside plate. | 6 / 16 x thickness of thinnest outside plate. | 1 | 7/8 | 8 x thickness of plate. |
| 5/8″ | 1 7/8 | 2 | 2 1/2 | 4 1/2 / " | " | 1 1/8 | 1 | " |
| 3/4″ | 2 1/4 | 2 1/2 | 3 | 5 / " | " | 1 1/4 | 1 1/8 | " |
| 7/8″ | 2 5/8 | 3 | 3 1/2 | 6 / " | " | 1 1/2 | 1 1/4 | " |

## TABLE 36.

### Old Standard Connections for Beams and Channels.
### American Bridge Company.

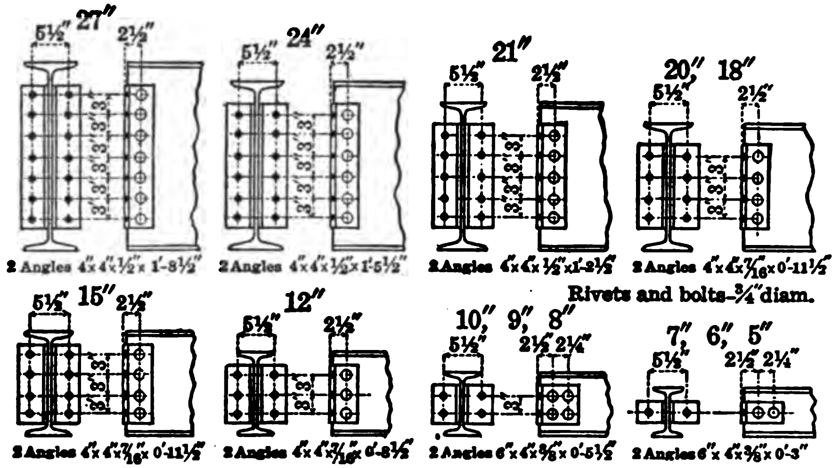













| Size | Two Angle Connections | One Angle Connections |
|---|---|---|
| 24" | 2 Ls 4"x4"x$\frac{7}{16}$"x1'-5½"<br>Weight 36 pounds | 1 L 6"x6"x$\frac{7}{16}$"x1'-5½"<br>Weight 30 pounds |
| 20"<br>18" | 2 Ls 4"x4"x$\frac{7}{16}$"x1'-2½"<br>Weight 30 pounds | 1 L 6"x6"x$\frac{7}{16}$"x1'-2½"<br>Weight 25 pounds |
| 15" | 2 Ls 6"x4"x$\frac{7}{16}$"x10"<br>Weight 27 pounds | 1 L 6"x6"x$\frac{7}{16}$"x10"<br>Weight 17 pounds |
| 12" | 2 Ls 6"x4"x$\frac{7}{16}$"x7½"<br>Weight 20 pounds | 1 L 6"x6"x$\frac{7}{16}$"x7½"<br>Weight 13 pounds |
| 10"<br>9"<br>8"<br>7" | 2 Ls 6"x4"x$\frac{7}{16}$"x5"<br>Weight 14 pounds | 1 L 6"x6"x$\frac{7}{16}$"x5"<br>Weight 9 pounds |
| 6"<br>5"<br>4"<br>3" | 6"&5": 2 Ls 6"x4"x$\frac{7}{16}$"x2½"<br>Weight 7 pounds<br>4"&3": 2 Ls 6"x4"x$\frac{7}{16}$"x2"<br>Weight 6 pounds | 6"&5": 1 L 6"x6"x$\frac{7}{16}$"x2½"<br>Weight 5 pounds<br>4"&3": 1 L 6"x6"x$\frac{7}{16}$"x2"<br>Weight 4 pounds |

Weights of connections include gross weights of angles and weights of ¾" shop rivets

# TABLE 37.

**New Standard Connections for Beams and Channels. American Bridge Company.**




**Limiting Values of Beam Connections.**

| I Beams. | | Value of Web Connection. | Values of Outstanding Legs of Connection Angles. | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Field Rivets. | | | Field Bolts. | | |
| Depth, Inches. | Weight, Lb. Per Foot. | Shop Rivets in Enclosed Bearing, Pounds. | ¾" Rivets or Turned Bolts, Single Shear, Pounds. | Min. Allowable Span in Feet, Uniform Load. | t, In. | ¾" Rough Bolts, Single Shear, Pounds. | Min. Allowable Span in Feet, Uniform Load. | t, In. |
| 27 | 83 | 66,800 | 61,900 | 18.4 | ⅝ | 49,500 | 23.1 | ⅝ |
| 24 | 80 | 67,500 | 53,000 | 17.5 | ⅝ | 42,400 | 21.9 | ⅝ |
| 24 | 69½ | 52,700 | 53,000 | 16.3 | ⅝ | 42,400 | 20.2 | ⅝ |
| 21 | 57½ | 40,200 | 44,200 | 15.5 | 9/16 | 35,300 | 17.6 | ⅝ |
| 20 | 65 | 45,000 | 35,300 | 17.6 | ⅝ | 28,300 | 22.1 | ⅝ |
| 18 | 55 | 41,400 | 35,300 | 13.3 | ½ | 28,300 | 16.7 | ⅝ |
| 18 | 46 | 29,000 | 35,300 | 15.0 | ½ | 28,300 | 15.4 | ⅝ |
| 15 | 42 | 36,900 | 35,300 | 8.9 | ⅝ | 28,300 | 11.1 | ⅝ |
| 15 | 36 | 26,000 | 35,300 | 11.1 | 7/16 | 28,300 | 11.1 | 9/16 |
| 12 | 31½ | 23,600 | 26,500 | 8.1 | 9/16 | 21,200 | 9.0 | ⅝ |
| 12 | 27½ | 17,200 | 26,500 | 10.3 | 7/16 | 21,200 | 10.3 | ⅝ |
| 10 | 25 | 27,900 | 17,700 | 7.4 | ⅜ | 14,100 | 9.2 | ⅝ |
| 10 | 22 | 20,900 | 17,700 | 6.9 | ⅜ | 14,100 | 8.6 | ⅝ |
| 9 | 21 | 26,100 | 17,700 | 5.7 | ⅜ | 14,100 | 7.1 | ⅝ |
| 8 | 18 | 24,300 | 17,700 | 4.3 | ⅜ | 14,100 | 5.4 | ⅝ |
| 8 | 17½ | 18,900 | 17,700 | 4.4 | ⅜ | 14,100 | 5.5 | ⅝ |
| 7 | 15 | 11,300 | 8,800 | 6.2 | ⅜ | 7,100 | 7.8 | ⅝ |
| 6 | 12¼ | 10,400 | 8,800 | 4.4 | ⅜ | 7,100 | 5.5 | ⅝ |
| 5 | 9¾ | 9,500 | 8,800 | 2.9 | ⅜ | 7,100 | 3.6 | ⅝ |

**Allowable Unit Stress in Pounds Per Square Inch.**

| | | | | |
|---|---|---|---|---|
| Single Shear | Rivets................Shop 12,000<br>Rivets and Turned Bolts . Field 10,000<br>Rough Bolts...........Field 8,000 | | Bearing | Rivets—enclosed..........Shop 30,000<br>Rivets—one side...........Shop 24,000<br>Rivets and Turned Bolts . . . Field 20,000<br>Rough Bolts...............Field 16,000 |

t = Web thickness, in bearing, to develop max. allowable reactions, when beams frame opposite.

Connections are figured for bearing and shear (no moment considered).

The above values agree with tests made on beams under ordinary conditions of use.

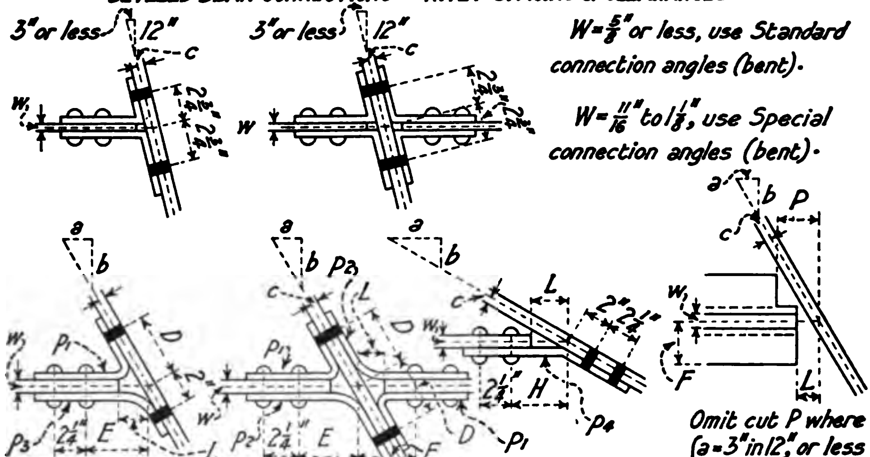
Where web is enclosed between connection angles (enclosed bearing), values are greater because of the increased efficiency due to friction and grip.

Special connections shall be used when any of the limiting conditions given above are exceeded—such as end reaction from loaded beam being greater than value of connection; shorter span with beam fully loaded; or a less thickness of web when maximum allowable reactions are used.

# TABLE 38.

## Standard Beveled Beam Connections.
## American Bridge Company.

BEVELED BEAM CONNECTIONS — RIVET SPACING & CLEARANCES



W = 5/8" or less, use Standard connection angles (bent).

W = 11/16" to 1 1/8", use Special connection angles (bent).

Omit cut P where a = 3" in 12", or less; c = 1/4" or less; F = 3" or less.

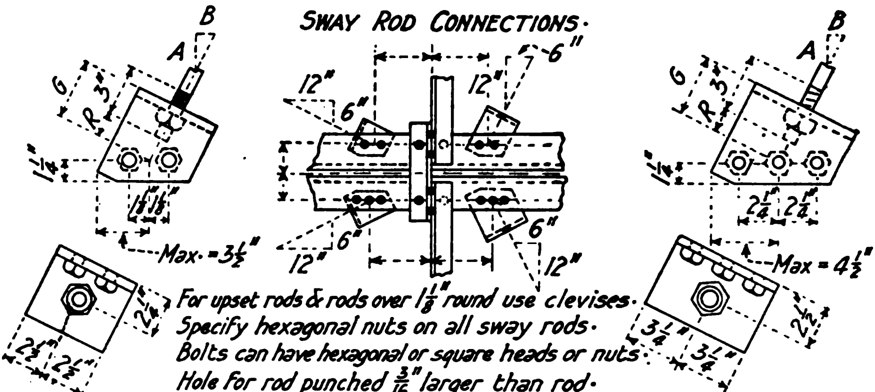
For large duplication modify these details where necessary to permit machine riveting. Table covers plates up to 1/2" thick.

| a | b | Max. c | Max. w | D | E | H | Length of Bent Plates $P_1$ | $P_2$ | $P_3$ | $P_4$ | L | P F = up to 3" | P F = 3" to 4" |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1" | 12" | 9/16" | 1 1/8" | 2 3/4" | See notes above. | | | | | | 1" | 1 1/2" | 1 1/2" |
| 2 | 12 | 9/16 | 1 1/8 | 2 3/4 | | | | | | | 1 1/4 | 1 3/4 | 1 3/4 |
| 3 | 12 | 9/16 | 1 1/8 | 2 3/4 | | | | | | | 1 1/4 | 2 | 2 1/4 |
| 4 | 12 | 9/16 | 1 1/8 | 3 1/2 | 3 1/2" | 2 3/4" | 10" | 11 1/2" | 10" | 12" | 1 1/4 | 2 1/4 | 2 1/2 |
| 5 | 12 | 9/16 | 1 1/8 | 4 | 4 | 3 | 11 | 12 1/2 | 10 1/2 | 12 | 1 1/2 | 2 1/2 | 3 |
| 6 | 12 | 9/16 | 1 1/8 | 4 1/2 | 4 1/2 | 3 | 12 | 13 1/2 | 11 | 12 | 1 1/2 | 2 3/4 | 3 1/4 |
| 7 | 12 | 9/16 | 1 1/8 | 5 | 5 | 3 1/4 | 12 1/2 | 14 1/2 | 11 1/2 | 12 | 1 3/4 | 3 | 3 1/2 |
| 8 | 12 | 9/16 | 1 1/8 | 5 1/2 | 5 1/2 | 3 1/4 | 13 | 15 1/2 | 12 | 12 | 1 3/4 | 3 1/4 | 4 |
| 9 | 12 | 9/16 | 1 1/8 | | | 3 1/4 | | | | 12 | 1 3/4 | 3 1/2 | 4 1/2 |
| 10 | 12 | 9/16 | 1 1/8 | | | 3 1/2 | | | | 12 1/2 | 2 | 4 | 5 |
| 11 | 12 | 9/16 | 1 1/8 | | | 3 1/2 | | | | 12 1/2 | 2 | 4 1/2 | 5 1/2 |
| 12 | 12 | 9/16 | 1 1/8 | | | 3 3/4 | | | | 12 1/2 | 2 1/4 | 4 1/2 | 5 1/2 |
| 12 | 11 | 1/4 | 1/2 | | | 2 3/4 | | | | 12 | 1 1/4 | 4 1/2 | 5 1/2 |
| 12 | 10 | 1/4 | 1/2 | | | 3 | | | | 12 | 1 1/2 | 5 | 6 |
| 12 | 9 | 1/4 | 1/2 | | | 3 | | | | 12 | 1 1/2 | 5 1/2 | 6 1/2 |
| 12 | 8 | 1/4 | 1/2 | | | 3 1/4 | | | | 12 | 1 3/4 | 6 | 7 1/2 |
| 12 | 7 | 1/4 | 1/2 | | | 3 1/2 | | | | 12 1/2 | 2 | 6 1/2 | 8 1/2 |
| 12 | 6 | 1/4 | 1/2 | | | 3 3/4 | | | | 12 1/2 | 2 1/4 | 7 1/2 | 10 |
| 12 | 5 | 1/4 | 1/2 | | | 4 | | | | 13 | 2 1/2 | 9 | 11 1/2 |
| 12 | 4 | 1/4 | 1/2 | | | 4 3/4 | | | | 13 1/2 | 3 1/4 | 11 | 14 |

## TABLE 39.

### Standard Sway Rod and Lateral Connections.
### American Bridge Company.

SWAY ROD CONNECTIONS.




For upset rods & rods over $1\frac{1}{8}$" round use clevises.
Specify hexagonal nuts on all sway rods.
Bolts can have hexagonal or square heads or nuts.
Hole for rod punched $\frac{3}{16}$" larger than rod.

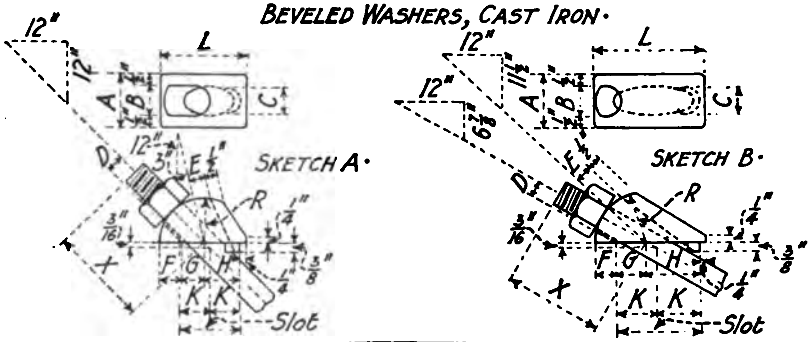
Rod $\frac{3}{4}$" round (not upset), Bolts $\frac{3}{4}$" or $\frac{5}{8}$" round.
Rod $\frac{7}{8}$" round (not upset), Bolts $\frac{3}{4}$" or $\frac{5}{8}$" round.

| A | B | Size of Angle | G | R |
|---|---|---|---|---|
| 12" | 6" to 12" | 6"×4"×$\frac{1}{2}$", 5" long | $3\frac{3}{4}$" | $1\frac{1}{2}$" |
| 6" to 12" | 12" | 6"×4"×$\frac{1}{2}$", 5 long | $3\frac{3}{4}$" | $1\frac{1}{2}$" |

Rod 1" round (not upset), Bolts $\frac{3}{4}$" or $\frac{5}{8}$" round.
Rod $1\frac{1}{8}$" round (not upset), Bolts $\frac{3}{4}$" round.

| A | B | Size of Angle | G | R |
|---|---|---|---|---|
| 12" | 6" to 12" | 6"×4"×$\frac{5}{8}$", $6\frac{1}{2}$" long | $3\frac{1}{4}$" | $\frac{3}{4}$" |
| 6" to 12" | 12" | 8"×4"×$\frac{5}{8}$", $6\frac{1}{2}$ long | $4\frac{1}{8}$" | 2" |

BEVELED WASHERS, CAST IRON.



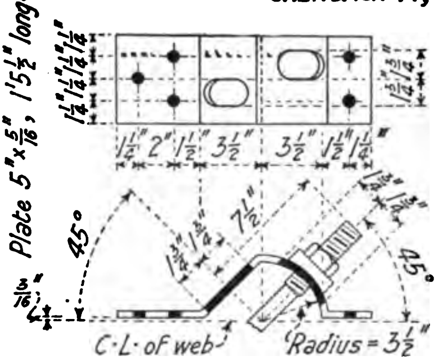

| Sketch | Round Rod | Upset | A | B | C | D | E | F | G | H | L | R | X | K | Size of Slot in Plate | Weight Pounds |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| A | $\frac{7}{8}$" 1" | None | $2\frac{1}{8}$" | $1\frac{1}{8}$" | 1" | $\frac{9}{16}$" | $\frac{9}{16}$" | $\frac{7}{8}$" | $\frac{7}{8}$" | $1\frac{3}{8}$" | $3\frac{3}{8}$" | $1\frac{3}{4}$" | 4" | $1\frac{1}{8}$" | $1\frac{1}{8}$"×$2\frac{1}{4}$" | 1.8 |
| A | 1 $1\frac{1}{8}$ | $1\frac{3}{8}$" $1\frac{1}{2}$ | $2\frac{5}{8}$ | $1\frac{5}{8}$ | $1\frac{1}{2}$ | $\frac{13}{16}$ | $\frac{13}{16}$ | $\frac{7}{8}$ | $1\frac{1}{8}$ | $1\frac{7}{8}$ | $4\frac{1}{8}$ | 2 | 5 | $1\frac{5}{8}$ | $1\frac{3}{4}$×$3\frac{1}{4}$ | 2.6 |
| B | $\frac{7}{8}$ 1 | None | $2\frac{1}{8}$ | $1\frac{1}{8}$ | 1 | $\frac{9}{16}$ | $\frac{9}{16}$ | $\frac{3}{4}$ | $1\frac{1}{4}$ | $2\frac{1}{4}$ | $4\frac{1}{2}$ | 2 | $4\frac{1}{2}$ | $1\frac{3}{4}$ | $1\frac{1}{8}$"×$3\frac{1}{2}$" | 2.3 |
| B | 1 $1\frac{1}{8}$ | $1\frac{3}{8}$ $1\frac{1}{2}$ | $2\frac{5}{8}$ | $1\frac{5}{8}$ | $1\frac{1}{2}$ | $\frac{13}{16}$ | $\frac{13}{16}$ | $\frac{3}{4}$ | $1\frac{7}{8}$ | $3\frac{1}{8}$ | 6 | $2\frac{5}{8}$ | 6 | $2\frac{5}{8}$ | $1\frac{3}{4}$×$5\frac{1}{4}$ | 3.8 |

For rods above $1\frac{1}{8}$" diam. use clevis connections.

## TABLE 40.

### Standard Lateral Connections for Highway Bridges.
### American Bridge Company.

SKEWBACK "A", Weight 6·8 lbs·



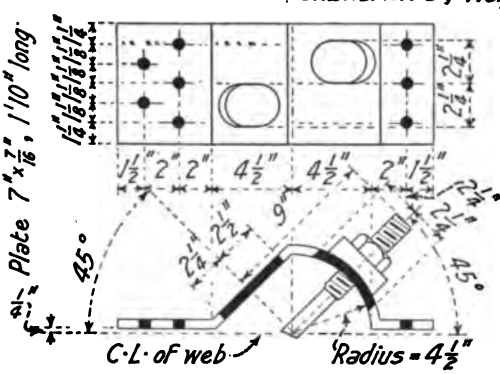

Skewback "A" for rods up to $1\frac{1}{4}$" round or $1\frac{1}{8}$" square (upset to $1\frac{5}{8}$" round);

For upsets $1\frac{1}{8}$" diam· or less, angle of rod may vary from 32° ($7\frac{1}{2}$" in 12") to 60° (12" in $6\frac{15}{16}$")·

For upsets greater than $1\frac{1}{8}$" diam· up to $1\frac{5}{8}$" diam·, angle of rod may vary from $41\frac{1}{2}$° ($10\frac{5}{8}$" in 12") to 60° (12" in $6\frac{15}{16}$")·

Standard slot in beam $3\frac{1}{2}$" × 6"·

SKEWBACK "B", Weight 17 lbs·



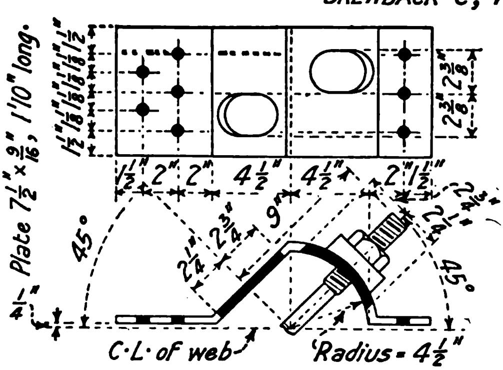

Skewback "B" for rods $1\frac{1}{4}$" round or $1\frac{1}{8}$" square (upset to $1\frac{5}{8}$" round); up to $1\frac{1}{2}$" round (upset to $1\frac{7}{8}$" round) or $1\frac{3}{8}$" square (upset to 2" round·)

For upsets $1\frac{5}{8}$" diam· or less, angle of rod may vary from $33\frac{2}{3}$° (8" in 12") to 60° (12" in $6\frac{15}{16}$")·

For upsets greater than $1\frac{5}{8}$" diam· up to 2" diam·, angle of rod may vary from $38\frac{2}{3}$° ($9\frac{19}{32}$" in 12") to 60° (12" in $6\frac{15}{16}$")·

Standard slot in beam $4\frac{1}{4}$" × $6\frac{1}{2}$"·

SKEWBACK "C", Weight 23 lbs·




Skewback "C" for rods $1\frac{9}{16}$" round or $1\frac{7}{16}$" square (upset to 2" round); up to $1\frac{3}{4}$" round (upset to $2\frac{1}{8}$" round) or $1\frac{1}{2}$" square (upset to $2\frac{1}{4}$" round)

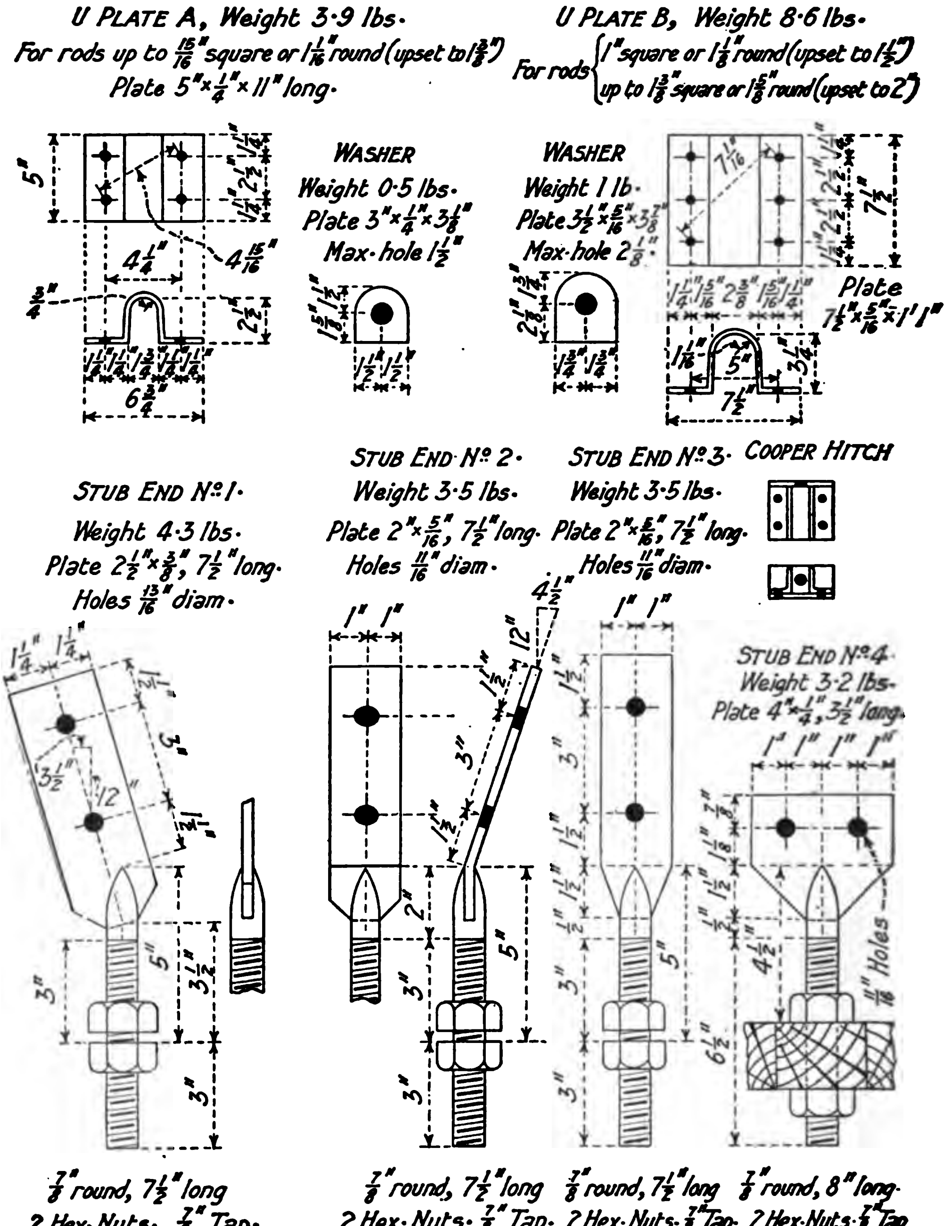
Angle of rod may vary from $40\frac{1}{2}$° ($10\frac{1}{4}$" in 12") to $64\frac{1}{3}$° (12" in $5\frac{3}{4}$") for all rods·

Standard slot in beam $4\frac{3}{4}$" × $6\frac{1}{2}$"

Where upset end of rod is greater than $2\frac{1}{8}$" diam·, hole in washer will be drilled to fit upset·

## TABLE 41.

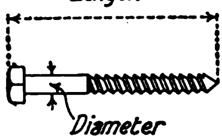
### Standard Lateral Connections and Stub Ends.
### American Bridge Company.



# TABLE 42.

**Standard Lag Screws, Hook Bolts and Washers.**
**American Bridge Company.**

## LAG SCREWS

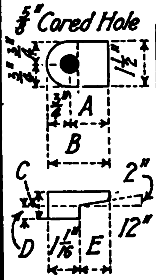



| Diam | Min. Length | Max. Length | No. Thread per inch |
|---|---|---|---|
| $\frac{5}{16}$" | $1\frac{1}{2}$" | 6" | |
| $\frac{3}{8}$ | $1\frac{1}{2}$ | 6 | |
| $\frac{7}{16}$ | $1\frac{1}{2}$ | 8 | |
| $\frac{1}{2}$ | $1\frac{1}{2}$ | 10 | |
| $\frac{9}{16}$ | 2 | 12 | |
| $\frac{5}{8}$ | 2 | 12 | 5 |
| $\frac{3}{4}$ | $2\frac{1}{2}$ | 12 | 4 |
| $\frac{7}{8}$ | 3 | 12 | 3 |
| 1 | $3\frac{1}{2}$ | 12 | |
| $1\frac{1}{8}$ | 5 | 12 | |
| $1\frac{1}{4}$ | 6 | 12 | |
| $1\frac{1}{2}$ | 8 | 12 | |

| Length of Lag Screw & Head | |
|---|---|
| Length of Screw | Length of Head |
| $1\frac{1}{2}$" | 1" |
| 2 | $1\frac{1}{4}$ |
| $2\frac{1}{2}$ | $1\frac{1}{2}$ |
| 3 | $1\frac{3}{4}$ |
| $3\frac{1}{2}$ | 2 |
| 4 | $2\frac{1}{4}$ |
| $4\frac{1}{2}$ | $2\frac{1}{2}$ |
| 5 | $2\frac{3}{4}$ |
| $5\frac{1}{2}$ | 3 |
| 6 | $3\frac{1}{4}$ |
| 7 | $3\frac{3}{4}$ |
| 8 | $4\frac{1}{4}$ |
| 9 | $4\frac{3}{4}$ |
| 10 | 5 |
| 11 | 5 |
| 12 | 5 |

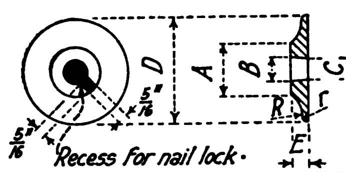
Heads are the same as for square head bolts.
Threaded portion is not tapered except at point

## BEAM CLAMP



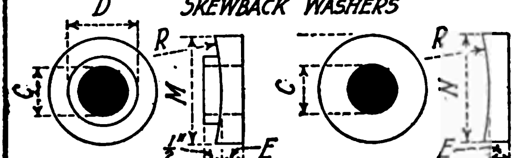

| Size Beam | Dimensions of Clamp A | B | C | D | E | Weight in lbs. |
|---|---|---|---|---|---|---|
| 18" | $1\frac{1}{2}$" | $2\frac{1}{4}$ | $\frac{7}{8}$" | $\frac{15}{32}$" | $1\frac{3}{16}$" | 0.4 |
| 15 | $1\frac{1}{2}$ | $2\frac{1}{4}$ | $\frac{7}{8}$ | $\frac{13}{32}$ | $1\frac{3}{16}$ | 0.4 |
| 12 | $1\frac{1}{2}$ | $2\frac{1}{4}$ | $\frac{7}{8}$ | $\frac{11}{32}$ | $1\frac{3}{16}$ | 0.4 |
| 9&10 | $1\frac{1}{2}$ | $2\frac{1}{4}$ | $\frac{3}{4}$ | $\frac{5}{16}$ | $1\frac{3}{16}$ | 0.4 |
| 7&8 | $1\frac{1}{4}$ | 2 | $\frac{3}{4}$ | $\frac{1}{4}$ | $\frac{15}{16}$ | 0.4 |
| 5&6 | $1\frac{1}{4}$ | 2 | $\frac{5}{8}$ | $\frac{7}{32}$ | $\frac{15}{16}$ | 0.3 |

## OGEE WASHERS



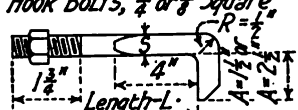

| Size Bolt | Dimensions of Washer A | B | C | D | E | R | r | Weight in Pounds |
|---|---|---|---|---|---|---|---|---|
| $\frac{5}{8}$" | $1\frac{3}{8}$" | $\frac{3}{4}$" | $\frac{11}{16}$" | $2\frac{3}{4}$" | $\frac{1}{2}$" | $\frac{5}{8}$" | $\frac{3}{16}$" | 0.4 |
| $\frac{3}{4}$ | $1\frac{5}{8}$ | $\frac{7}{8}$ | $\frac{13}{16}$ | $3\frac{1}{4}$ | $\frac{5}{8}$ | $\frac{3}{4}$ | $\frac{7}{32}$ | 0.7 |
| $\frac{7}{8}$ | $1\frac{7}{8}$ | $1\frac{1}{16}$ | $\frac{31}{32}$ | $3\frac{3}{4}$ | $\frac{3}{4}$ | $\frac{7}{8}$ | $\frac{1}{4}$ | 1.0 |

## SKEWBACK WASHERS



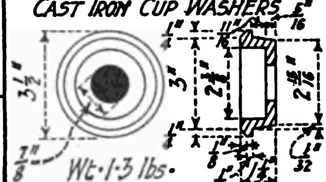

| Used With | Dimensions of Washers M | N | C | D | E | R | Weight in Pounds |
|---|---|---|---|---|---|---|---|
| Skewback "A" | $2\frac{3}{4}$" | | $1\frac{1}{4}$" | $1\frac{5}{8}$" | $\frac{3}{4}$" | $3\frac{13}{16}$" | 1.2 |
| | | $3\frac{1}{4}$" | $1\frac{1}{2}$ | | $\frac{7}{8}$ | $3\frac{13}{16}$ | 1.8 |
| | | $3\frac{3}{4}$ | $1\frac{3}{4}$ | | 1 | $3\frac{13}{16}$ | 2.5 |
| Skewback "B" | $3\frac{3}{4}$" | | $1\frac{3}{4}$" | $2\frac{1}{8}$" | 1" | $4\frac{15}{16}$" | 2.7 |
| | | 4" | 2 | | 1 | $4\frac{15}{16}$ | 3.0 |
| | | $4\frac{1}{2}$ | $2\frac{1}{4}$ | | 1 | $4\frac{15}{16}$ | 3.9 |

## HOOK BOLTS, $\frac{3}{4}$" or $\frac{7}{8}$" Square,



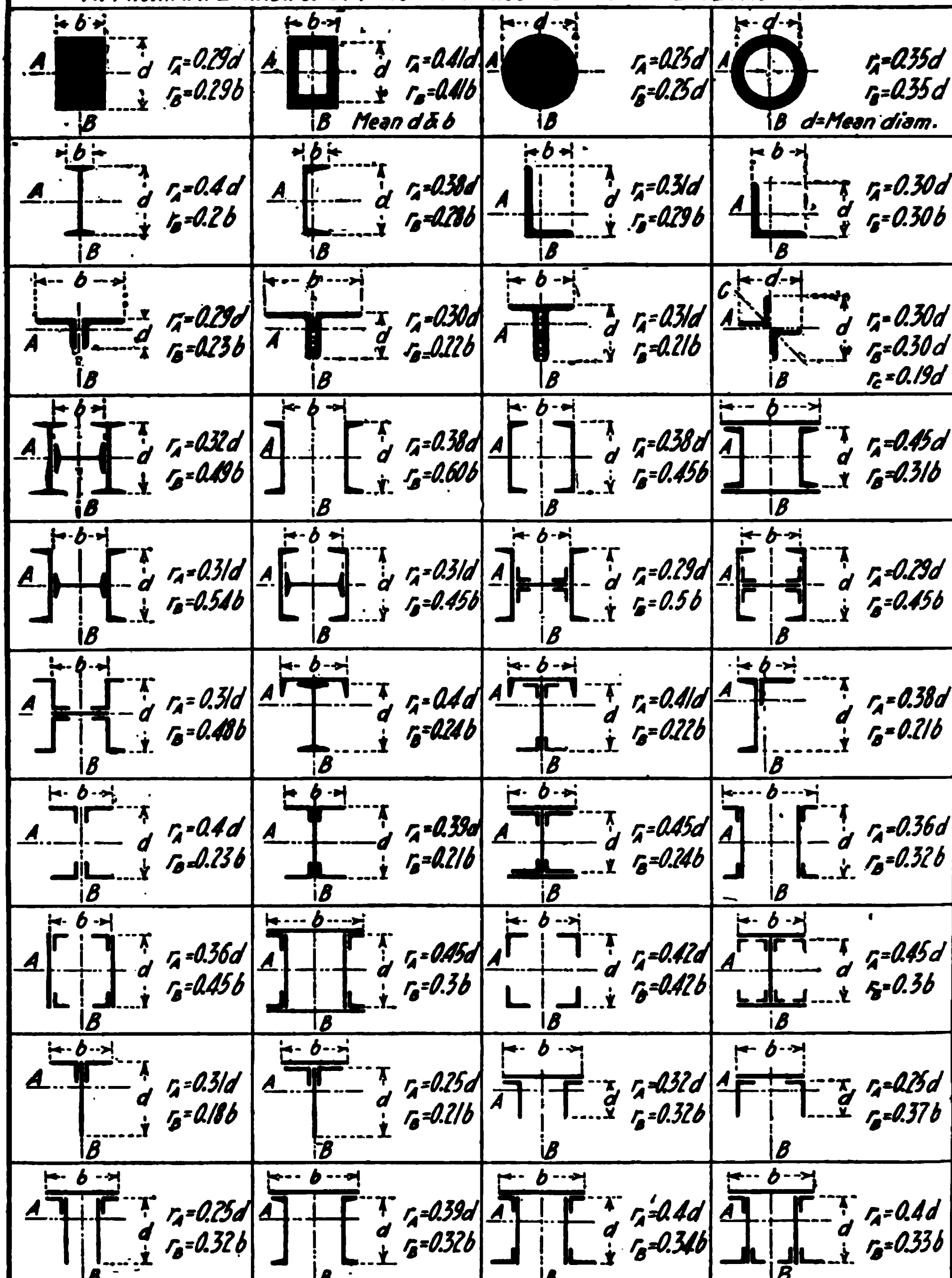

In billing Hook Bolts give dimensions A, S & L; all other dimensions are standard. Unless otherwise specified, "S" will be made $\frac{3}{4}$". Hex. nuts furnished.

## CAST IRON CUP WASHERS



## TABLE 43.

### APPROXIMATE RADII OF GYRATION OF VARIOUS STRUCTURAL SECTIONS.



| | | | |
|---|---|---|---|
| $r_A=0.29d$, $r_B=0.29b$ | $r_A=0.41d$, $r_B=0.41b$; Mean d & b | $r_A=0.25d$, $r_B=0.25d$ | $r_A=0.35d$, $r_B=0.35d$; d=Mean diam. |
| $r_A=0.4d$, $r_B=0.2b$ | $r_A=0.38d$, $r_B=0.28b$ | $r_A=0.31d$, $r_B=0.29b$ | $r_A=0.30d$, $r_B=0.30b$ |
| $r_A=0.29d$, $r_B=0.23b$ | $r_A=0.30d$, $r_B=0.22b$ | $r_A=0.31d$, $r_B=0.21b$ | $r_A=0.30d$, $r_B=0.30d$, $r_C=0.19d$ |
| $r_A=0.32d$, $r_B=0.49b$ | $r_A=0.38d$, $r_B=0.60b$ | $r_A=0.38d$, $r_B=0.45b$ | $r_A=0.45d$, $r_B=0.31b$ |
| $r_A=0.31d$, $r_B=0.54b$ | $r_A=0.31d$, $r_B=0.45b$ | $r_A=0.29d$, $r_B=0.5b$ | $r_A=0.29d$, $r_B=0.45b$ |
| $r_A=0.31d$, $r_B=0.48b$ | $r_A=0.4d$, $r_B=0.24b$ | $r_A=0.41d$, $r_B=0.22b$ | $r_A=0.38d$, $r_B=0.21b$ |
| $r_A=0.4d$, $r_B=0.23b$ | $r_A=0.39d$, $r_B=0.21b$ | $r_A=0.45d$, $r_B=0.24b$ | $r_A=0.36d$, $r_B=0.32b$ |
| $r_A=0.36d$, $r_B=0.45b$ | $r_A=0.45d$, $r_B=0.3b$ | $r_A=0.42d$, $r_B=0.42b$ | $r_A=0.45d$, $r_B=0.3b$ |
| $r_A=0.31d$, $r_B=0.18b$ | $r_A=0.25d$, $r_B=0.21b$ | $r_A=0.32d$, $r_B=0.32b$ | $r_A=0.25d$, $r_B=0.37b$ |
| $r_A=0.25d$, $r_B=0.32b$ | $r_A=0.39d$, $r_B=0.32b$ | $r_A=0.4d$, $r_B=0.34b$ | $r_A=0.4d$, $r_B=0.33b$ |

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